APPLICATION OF

ROCK MECHANICS

IN SURFACE AND UNDERGROUND MINING

GOVERNMENT OF INDIA
MINISTRY OF MINES
INDIAN BUREAU OF MINES

Government of India
Ministry of Mines
INDIAN BUREAU OF MINES

Application of Rock Mechanics in Surface and Underground Mining

Issued by
Controller General
Indian Bureau of Mines

54/500/June 2014

Indian Bureau of Mines

Published by
C.S. Gundewar, *Controller General*

General Guidance	**C.S. Gundewar,** *Controller General*
Formulation & Script Review	**Dr B.P. Sinha,** *Controller of Mines (TMP) (Rtd)*
Drafting & Compilation	**Dr A. Santharam,** *Regional Mining Geologist (Rtd)*
Technical Editing	**S.R. Roy,** *Regional Controller of Mines*
Publication Review & Coordination	**A.K. Singh,** *Chief Editor*
General Editing & Layout Design Control	**M. Sumesh,** *Senior Editor*
Production & Printing	**Indira S. Nair,** *Assistant Editor* **A.A. Gawai,** *Senior Technical Assistant* **P.L. Masram,** *Senior Technical Assistant* **Anurag P. Mishra,** *Junior Technical Assistant*

Conceptualised & Prepared by
Technical Consultancy, Mining Research & Publication Division

Formatted & Produced at
Publication Section, ME Division

Printed at
IBM Press, Indian Bureau of Mines Headquarters, Nagpur

First Published **2014**

This Publication is available at **< http://ibm.gov.in/>**

For more information contact **Indian Bureau of Mines, Indira Bhavan, Civil Lines, Nagpur, Maharashtra, India**

Preface

Rock Mechanics is a budding branch of Mining Engineering and Goesciences which in todays context has been gaining wide attention, momentum and acceptance by the Mining Fraternity. This Publication entitled " Bulletin on Application of Rock Mechanics in Surface and Underground Mining" is an endeavour to comprehensively assimilate the scope and reach of this emerging Engineering discipline. The design of rock excavations is predominantly based on past experiences, engineering judgments and empirical methods. Rock masses invariably are heterogeneous and vary in their engineering properties and are intersected persistently by discontinuity planes. It is, therefore, a challenge to create realistic simulations of excavation in rock, which is not only unpredictable but also very complex as there are no simplistic assumptions by which rock mass diversities could be explained.

The present scenario of rapidly expanding mining and civil engineering sectors calls for scientific approach in formulating designs of opencast, underground and civil excavations. Recent years have seen numerical modelling techniques and softwares replacing the traditional methods of rock excavation design.

The major premise on which this Bulletin was conceptualised is to proffer an exposition of academic and reference value to the Mineral Industry professionals. Dealing profusely on topics related to application of rock mechanics in opencast and underground mining operations, this Bulletin throws insights about the techniques of rock mechanics and their utilisation in improving productivity and safety in mines. Aimed at providing a holistic content about rock mechanics principles and their applications in safe and systematic mining, mine economics and conservation of minerals, this Bulletin attempts to elaborate not only on the fundamentals of Rock Mechanics but also expatiates in detail the topics, such as, Rock Mass Classification, Slope Stability, Numerical Methods in Rock Excavation & Designs and Application of Rock Mechanics in Opencast & Underground Mines. The overall composition of the subject literature oscillate from moderate to extensive and encompasses some hypotheses and experimental results. The mathematical aspects have been intentionally kept to minimum to engender lucidity to the text.

The discipline of Rock Mechanics has metamorphosed from rudimentary form in its nascent years to an advanced branch of study. Keeping in tandem with this tranformation, it is believed that this Bulletin will enable broadening the understanding of excavation design, planning of opencast and underground mine and the scientific principles that govern such concepts which would further engender improved recovery of minerals with safety imperatives duly in place. Though intended as ready reference handbook for academicians, researchers and field personnel involved in mining operations, it is hoped that this Bulletin would serve the best interest of all its readers.

Nagpur
Dated : 01 January '14

Controller General
Indian Bureau of Mines

Contents

List of Tables

List of Figures

List of Figures

List of Figures

Chapter

1 Background of Rock Mechanics

1.0 INTRODUCTION

In 1951, the First International Convention on Rock Pressure and Ground Support was held in Liege, Belgium and after this, several other national and international Symposia and Conventions were held on the subject. In recent years, Numerical Methods are widely used to solve the problems related to rock mechanics.

Initially, the International Society for Rock Mechanics organised its first Conference on Rock Mechanics in 1966 at Lisbon. Definition of rock mechanics was described by the Committee on Rock Mechanics of Geological Society of America in 1964 and this was followed by the Committee on Rock Mechanics of National Academy of Sciences in 1966.

Rock Mechanics is the theoretical and applied science of behaviour of rock. It is that branch of mechanics, which is concerned, with the response of rock to the field of its environment. These techniques apply to surface excavation as well as underground. It has important role in mine planning and design viz. selection of mining methods, optimum slope angle, design of support system, drilling and blasting parameters.

The International Society for Rock Mechanics (ISRM) brought out its "Blue Book" containing all the ISRM Suggested Methods entitled "The Complete ISRM Suggested Methods for Rock Mass Characterisation, Testing and Monitoring (1974-2006) edited by R. Ulusay and J.A. Hudson in 2007. Considerable works on Rock mechanics and its applications related to opencast, underground mining, tunnelling, design of tailings dam and impoundments that were carried out on sound principles of rock mechanics engineering world wide along with important aspects of rock mechanics in slope engineering for slope stability analysis, slope stabilisation, design and improving the productivity were encompassed in this compilation of Suggested Methods (SMs).

1.1 BLASTING

Rock Blasting is one of the key areas that directly influences the productivity of mines as large quantities of well-fragmented ROM are required for the processing plants. It is desirable to minimise damage desired to rock slope after excavation of materials. The production of well-fragmented rock facilitates the post-excavation stages, such as, loading, transportation, handling and crushing. These requirements are possible only if proper blasting techniques are adopted & applied so as to control rock fragmentation and the consequent damages effected. Proper control of factors, such as, type, weight and distribution of explosives, blast hole diameter, effective burden, effective spacing, blast hole inclination, stemming, initiation sequence for detonation of explosives, delay between successive sequence hole or row firing etc. are essential for achieving optimum results. Rock Mechanics techniques that are based on rock mass characterisation studies help in selection of blasting parameters for specific rock mass.

1.2 GROUND VIBRATION STUDY

The ground vibration studies are helpful in designing blasting parameters, in order to reduce the distance of fly rocks and improved fragmentation. Blast induced vibrations are measured with the help of blasting seismograph. Depending on the site's ground conditions, an equation may be established to calculate a safe charge weight per delay and safe distance for measuring the ground vibration due to blasting within the permissible safe limits so as to avoid possible damage to important surface structures, such as, railway lines, crushers, buildings, archeological sites, temples and village localities. All these would have to be in compliance with the various regulations of the Mining Act.

1.3 GROUND CONTROL / STRATA CONTROL

Many underground mines have problems of stope design, ground control and support systems. For the purpose of analysis of the stability of existing pillars so as to avoid possible impending failure and to achieve safer designs of future stopes and pillar, geotechnical study is required. By using various instruments and close monitoring, stability of crown and rib pillars could be achieved.

1.4 SELECTION OF SUPPORT SYSTEMS

Geo-technical studies form an integral part for the assessment of ground conditions for support requirement. The geomechanics classification of rock mass in drives, stopes on the basis of rock mass classification, stand-up time for open stope, tunnel or without providing support can be studied. Type of supports and its density can be calculated.

1.5 DESIGN OF STOPE & PILLAR

Rock mechanics investigations are helpful for— a) design of different pillars in the stope, such as, rib pillar, barrier pillars, pillars against waterlogged area; b) design of stopes; c) the stability of stopes and various pillars—barrier pillars, shaft pillars etc.; and d) problem on ground control and design of support system. The major sources of instability of underground working are— a) adverse geological structure; b) excessively high rock stress while mining is at great depth; c) weathering or swelling of rock; d) excessive ground water pressure. Feasibility study for selecting underground mining methods require the rock mechanics data, such as, in situ stress, physio mechanical and elastic properties of rock and rock mass strength. For this,

a) The theoretical and experimental knowledge of stress distribution around the rock structure may be useful for interpreting the early failure of rock. Based on these, remedial measures could be taken which would enable saving in labour and cost. For planning and development of new deposits, this process help providing rational approach for designing the underground openings and supports.

b) For new deposits, the pre-existing state of stress usually assumed to be due to weight of overlaying rock would need to be assessed. Properties of subsurface rock can be obtained by laboratory testing core.

c) For underground design, an assessment of a small specimen of rocks that which would reveal the mechanical defects, such as, joints, fractures, and faults, which affect the in situ mechanical properties of the rock would be a valued input.

d) The mine planning not only attaches significant importance to stability of underground structure but also cost involved for sustenance of the mine during its projected lifetime.

e) Rock mechanics instruments commonly used for the purpose of evaluating the stability of working places, monitoring the ground movement, stress vibration etc. signal advance warning of impending instability so that safety of men and machinery could be ensured. Installation of such instruments is vital.

f) Monitoring of crown and rib pillar stability using vibrating wire stress meter, extensometer, strain and stress gauges; Determination of in situ stresses by using over coring technique, hydro fracturing techniques; Measurement of deformation using borehole deformation gauges; and Micro seismic network, are used for prediction of ground stability and seismic activity in mines that are very deep.

g) In order to analyse the stability of existing pillar so as to get a warning of possible failures and to achieve safer design of future stope and pillars, there is need to measure the in situ rock stresses at different levels in the mine together with mapping of joints and discontinuities for the evaluation of rock mass quality. The most popular and accepted classification system for rock mass strength is Q system. The value of the Q for a particular site is calculated from rock quality designation (RQD based on drill core samples) and from the nature of joint and stress reduction factors. It is helpful for deciding the support system at that particular site.

h) For designing underground workings the important parameters are spacing of joints and bedding planes, strike and dip of joints, continuity of joints and filling materials of joints. Core Orienters for measuring of joints, seismograph for ground vibration measurements, roof stability tester to evaluate the ground stability and continuous recording instruments should be installed.

i) Numerical modelling techniques have developed as a powerful tool for simulation of excavation and rock characteristics. This is very much helpful for designing of pillars, span of excavations and sequence of mining etc.

j) Unsupported roof over the underground openings may be massive or arched, laminated or layered. The arched roof usually form in massive rock while flat laminated roof is usually formed in sedimentary rocks. The degree of stress and strain in the roof, sidewall and floor of the opening can be evaluated by the elastic theory or measured by in situ methods.

1.6 SLOPE STABILITY INVESTIGATIONS

Nowadays, the number of opencast mines are increasing as compared to underground mines due to excessive demand for large volumes of ore and better ways available for waste handling, low gestation period and quick return on investments. Slope stability analysis is being carried out at the start-up of the project in order to avoid any further unexpected consequences. Design of ultimate pit slope angles, working bench height, bench slope angles, blast design parameters, selection of proper excavation and loading equipment invariably are dependent on the rock mass characterisation. The design of haul roads, ramps, foundation for crusher site, site evaluation for creating waste dumps, tailings dam facilities also require considerable amount of rock mechanics/soil mechanics investigations.

Slope support and stabilisation measures have been carried out to understand the behaviour of various geotechnical rock properties under different natural climatic conditions. Recent developments in numerical modelling techniques and monitoring instruments further proved helpful for the scientific mine design for improved recovery of minerals with safety and environmental considerations.

● ● ●

Chapter

2 Fundamentals of Rock Mechanics

2.0 INTRODUCTION

Rock mechanics engineering is the branch of engineering concerned with mechanical properties of rock and application of this knowledge in dealing with engineering problems of rock materials. Underground structures in rock, i.e., any excavated or natural subsurface opening or system of openings that is virtually supported by wall pillar only and not by any support placed within the openings need geotechnical study. For designing and stability evaluation— a) the stresses and/or deformation in the structure resulting from external or body load; b) the ability of structure to withstand the stress or deformation, need to be determined.

The behavior of fractured media is a complex problem that raises important issues of scale. Figure 2.1 (a) illustrates a fundamental problem faced when representing a fractured medium. The joint patterns, such as, blocky, irregular, tabular and columnar block shapes of rock mass are shown in Figure 2.1(b). Different types of joints sets are shown in Figure 2.1(c).

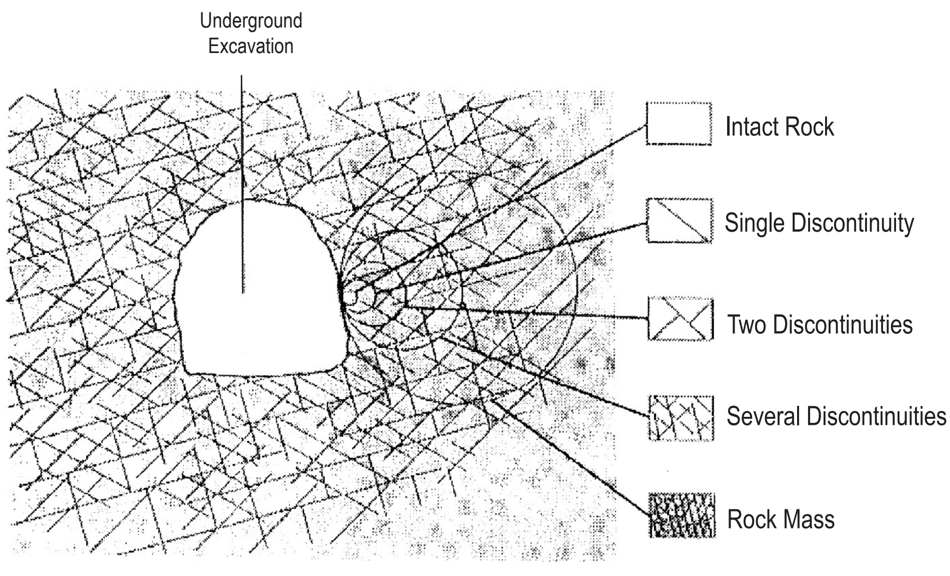


Figure 2.1(a): *Concept of Scale as it Pertains to Rock Mass Properties in Underground Excavation*

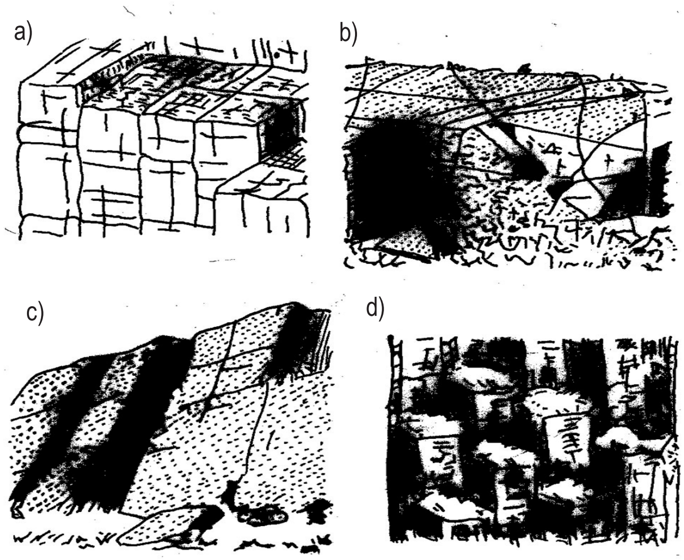


***Figure 2.1(b)**: Typical Joint Patterns*

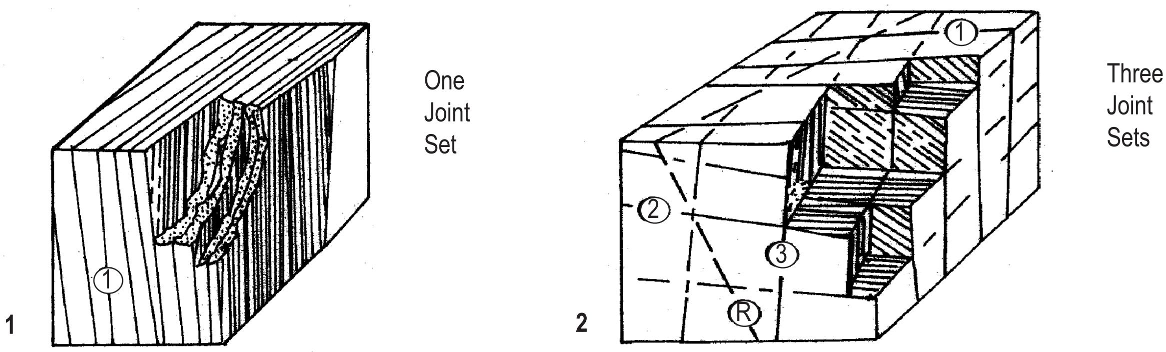


***Figure 2.1(c)**: Example of the Type and Influence the Number of Joints Sets Exert on Rock Mass*

The difficulties in designing a structure in rock are as follows:

a) State of stress of subsurface rock is not known due to weight of overlaying rock and tectonic forces.

b) The information on the mechanical properties of in situ rocks is not available before underground excavation.

c) The problem that arises in relation to calculating the stress and deformation in various parts of the rock structure.

The factors to deal with while designing a structure in rock are:

a) The design of the structure must be such that it holds up for a long span of time, all safety factors should be kept high.

b) Design should be such that the structure could be made in a minimum time.

c) In situ stress should be carried out at initial stage, so that modification at later stage could be avoided.

d) For economics of mining it is necessary to exploit the entire deposit keeping size and number of pillars, barriers to the minimum.

2.1 STRESS

There are two types of forces — a) *Body force* and b) *Surface force*. Body force acts through out the body and does not require physical contact with other body, i.e., gravitational, magnetic or inertial forces. Surface force acts on the external surface of a body and result from physical contact with other body. The term *stress means force per unit area*.

The rock specimen having geological discontinuity, such as, joints when under the loading system causes sliding along the discontinuity. The shear stress required causing sliding increases with increase of normal stress. The slope of the line relating to normal stress σ and shear stress τ defines the angle of friction ϕ. If the discontinuity surface is cemented or it is rough, a finite value of shear stress will be required to cause sliding when the normal stress is zero. This initial value of shear strength defines the cohesive strength *c* of the surface. The relation between shear stress and calculation of normal stress is shown in Figures 2.2 (a) & 2.2 (b).

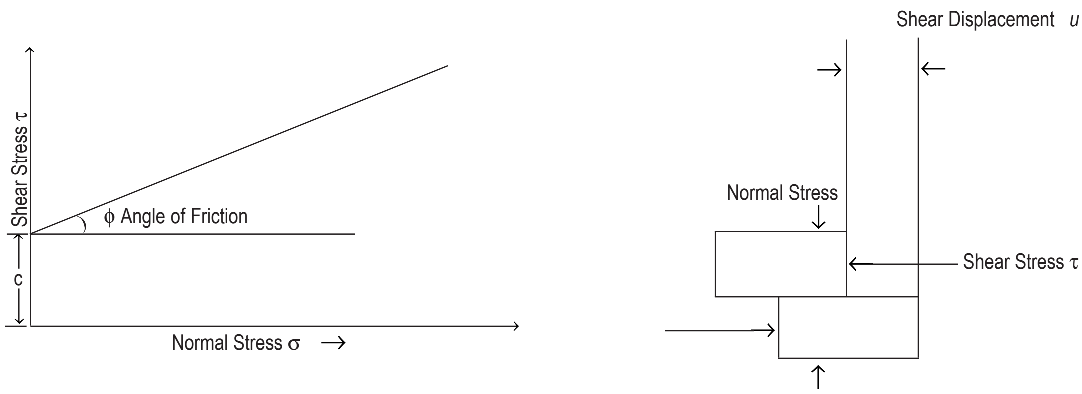


***Figure 2.2(a)**: Relation between Shear Stress, Normal Stress, Cohesion, Angle of Friction*

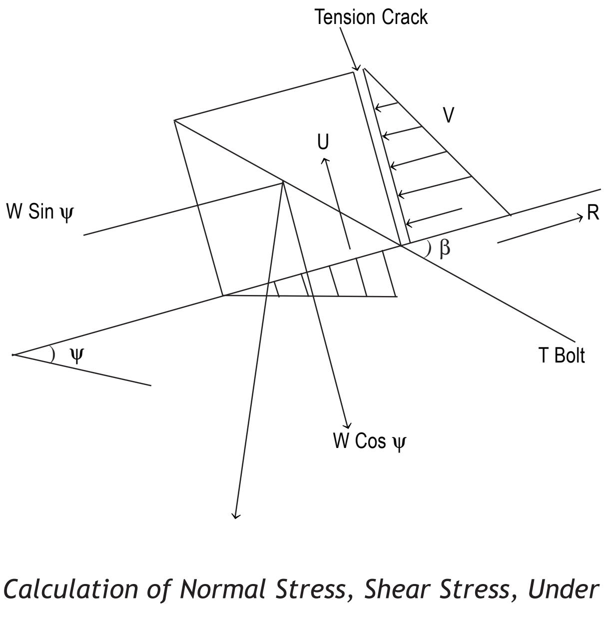


***Figure 2.2 (b)**: Calculation of Normal Stress, Shear Stress, Under Water Pressure*

When the block is on the slope, i.e., block having weight W, area A resting on the slope ground surface inclined at angle ψ then,

Normal stress $\sigma = (W \cos \psi) / A$

Shear stress $\tau = c + [(W \cos \psi) / A] \times \tan \phi$

Resisting force $R = \tau A = cA + W \cos \psi \tan \phi$

Uplift force $U = uA$, where, u = water pressure

Effect of water pressure Resisting force = $(W \cos \psi - U) \tan \phi$

When the tension crack is filled up with water, the water pressure increases linearly with depth. Then the equilibrium for block

$W \sin \psi + V = cA + (W \cos \psi - U) \tan \phi$

When bolting is used for stabilising the block sliding along the slope, bolt tension reduces the disturbing forces acting down the plane and increases the normal force and the frictional resistance between the base of the block and the plane. Then the factor of safety *F* defines the ratio of the total force available to resist sliding to total force tending induce sliding.

$$F = \frac{cA + (W \cos \psi - U) + T \cos \beta \tan \phi}{W \sin \psi + V - T \cos \beta}$$

When the slope is stable, the resisting forces are greater than the disturbing forces and the value of factor of safety will be greater than unity.

2.2 STRAIN AND YIELD

Formation of planes of separation in the rock result in fracture. Peak strength is the maximum stress reached before failure. It is indicated by Point 2 in the Figure 2.3 (a). Beyond the peak strength, the rock may still have some strength. The minimum or residual strength reached after post peak deformation is indicated by Point 3 in the Figure 2.3 (a). Yield occurs when there is a departure from elastic behaviour, i.e., when some of the deformation becomes irrecoverable at Point '1' in the Figure 2.3 (a). The yield stress σy in Figure 2.3 (a) is the stress at which permanent deformation first appears. Sudden loss of strength that occurs across a plane following little or no permanent deformation (plastic deformation) result in Brittle fracture as shown in Figure 2.3 (a). Ductile deformation occurs when rock may sustain further permanent deformation without loosing load carrying capacity as shown in Figure 2.3 (b).

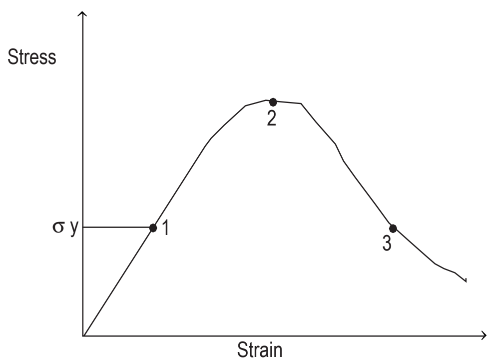


Figure 2.3(a): *Strain Softening*

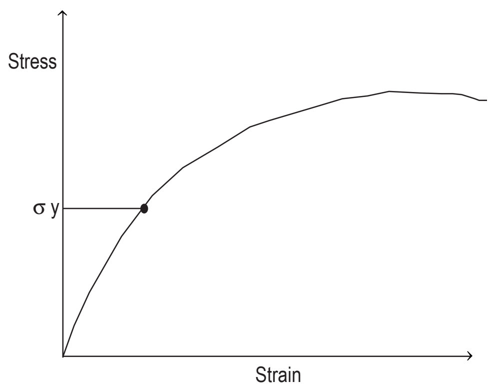


Figure 2.3(b): *Strain Hardening*

2.3 CREEP

Creep is time dependant property of testing of rock. Creep theory is used in design of pillar, slow deterioration or closure of mine working.

Creep deformation (or strain) is plotted against time and is shown in Figure 2.4. When constant stress is applied to the rock material, instantaneous elastic strain appears and curve is concave downward, creep in this region, i.e., (i) is called the *primary or transient creep*. The second region (ii), i.e., the steady state creep, is characterised by a curve of approximately constant slope. It is also called secondary creep. Finally, the curve as shown as (iii) becomes convex leading rapidly to fracture, this is called tertiary or accelerating creep.

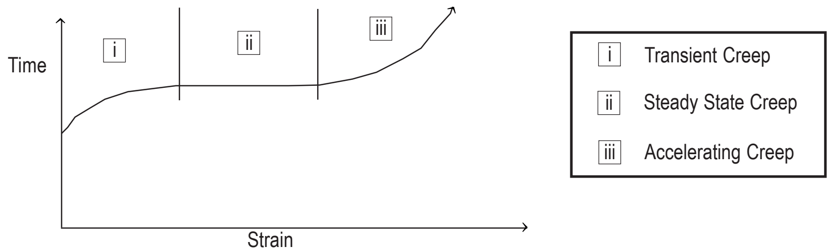


Figure 2.4: *Time-Strain Plot for Creep*

2.4 ELASTICITY AND MATERIAL PROPERTIES

Elastic theory provides the solution to a large number of problems that have direct applications in structure of rock mechanics. Many materials that are linearly elastic at low stress level deviate if they are put under higher stress or at prolonged loading system. The deviation may indicate an incomplete instantaneous recovery of strain upon removal of stress and variation in mechanical properties with application of stress. The behaviour of raw materials beyond the elastic range is termed as inelastic. The theory of elasticity can be used to make approximation of the stresses, strains and deformations in the structure under the given loading condition and that various inelastic theories can be used to estimate the ultimate loads that a given structure can support before there are occurrances of excessive deformation, fracture or disintegration. Inelastic theories have been developed mainly to describe the mechanical behaviour of ductile, plastic and viscous solids at high stress where creep and permanent deformation occur in rocks.

Most ductile materials have a fairly long plastic range before fracture occurs whereas brittle materials, such as, rocks have very limited plastic ranges, if any, for uniaxial tension or compression. However, for triaxial loading conditions, most brittle material tends to show a plastic range where permanent deformation occurs.

Theory of elasticity was developed by assuming that stress is directly proportional to strain. Various mathematical and mechanical models have been proposed to describe the mechanical behaviour of a real material. Some of the models are described below.

2.5 POISSION RATIO AND MODULUS OF ELASTICITY

Hooks law represents the relationship between stress and strain. Assume a rectangular parallel piped with its size parallel to coordinate axes acted upon by a normal stress σx uniformly distributed over two opposite sides, the magnitude of normal strain εx is given by

$$\varepsilon x = \frac{\sigma x}{E}$$

where, E is the Modulus of Elasticity

Extension of the body in the x direction is accompanied by a lateral contraction in both the y and z directions, thus

$$\varepsilon y = -v\frac{\sigma x}{E} \quad \text{and}$$

$$\varepsilon z = -v\frac{\sigma x}{E}$$

where v is a constant known as *Poisson's ratio.*

Poisson's ratio for many materials is between 0.15 and 0.35 and often assumed as equal to 0.25.

2.6 MECHANICAL PROPERTIES OF THE ROCK

For the purpose of design and to evaluate the stability of underground structure, mechanical properties of the rock must be known. It provides the knowledge of material deform or fail, under the action of applied force. The mechanical properties are tensile strength, compressive strength, shear strength, creep or time properties and strain or deformation properties. The mechanical properties can be determined by static testing which includes uniaxial (unconfined) compressive, tensile, shear and flexural strength, triaxial compressive, shear strength etc. and also elastic constants, i.e., modulus of elasticity and Poisson's ratio obtained from uniaxial, triaxial stress-strain relationship.

2.6.1 Uniaxial Compressive Strength

The uniaxial compressive strength of rock is measured by loading a cylindrical specimen to its failure in a compressive machine.

$$C_o = \frac{C_p}{0.778 + 0.222D/L}$$

where,

C_o is the compressive strength of a specimen of the same material having 1:1 length to diameter ratio.

C_p is compressive strength of specimen for which 2 > (L/D) > (1/3).

D is diameter of cylindrical samples and side length in case of a cubical sample.

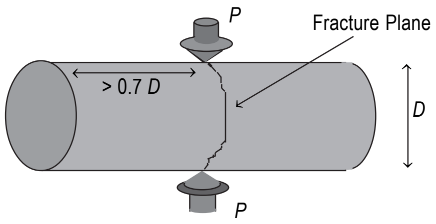
L is length or height of sample.

The factors, such as, flatness of bearing surface, a specimen size and shape, moisture content in the specimen, the effect of friction between the bearing platens and the specimen, the alignment of swivel head and rate of loading affect the test conducted for determining the compressive strength of a material.

The specimen generally must be cylindrical or cubical in shape. The cylindrical samples are cut to the size by a diamond saw and surface irregularities are smoothened by surface polishing machine. The length of the specimen is generally 2.5 times the diameter. The ends of the specimen should be parallel to each other and normal to the axes of specimen.

In underground mining, pillars and columns support the roof rock. For the stability of pillars and columns, the compressive strength of rock is a vital parameter.

The compressive strength of rock depends upon shape, surface quality of loading system, rock specimen surface, porosity and moisture content of the rock, rate of loading and specimen size. The compressive strength of the rock decreases with increase in its porosity. Water in rock pores reduces the magnitude of internal friction of rock thereby reducing the rock strength. Usually, wet sample has its strength 1/3 of that of a dry sample (Refer Figure 2.5).



For D = 50 mm $\qquad \sigma_c \approx 24\, I_s$

$$I_s = \frac{P}{D^2}$$

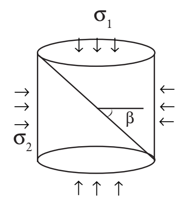
Failure criteria for isotropic and fractured rock

For other diameters (Bieniawski) : $\sigma_c \approx (14 + 0.175\,D) I_s$

D [mm]

- Failure of isotropic rock

$$\sigma_1 = \frac{2c \cos\varphi + \sigma_2 (1 + \sin\varphi)}{1 - \sin\varphi}$$



- Failure over pre-existing discontinuities

$$\sigma_1 = \frac{2c + \sigma_2 [\sin 2\beta + \tan\varphi\, (1 - \cos 2\beta)]}{\sin 2\beta - \tan\varphi\, (1 + \cos 2\beta)}$$

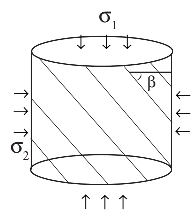


***Figure 2.5**: Uniaxial Compressive Strength Test*

2.6.2 Tensile Strength of Rock

The tensile strength is measured by loading a cylindrical specimen in tension to its failure. The indirect methods such as Brazilian test and flexural strength or bending test are also used to measure tensile strength.

a) Brazilian Test

The specimen is cut out of a cylindrical core by a diamond saw. The length to the diameter ratio is usually 0.5. The periphery of the specimen should be smooth. The specimen is placed under compression testing machine. Compressive load (normally 220 kgf/s) is applied slowly till failure take place (Refer Figure 2.6).

$$T_o = \frac{2F_c}{\pi DL}$$

F_c = applied failure load in kg along the length of the specimen

D = diameter of specimen in cm

L = length of the specimen in cm

T_o = uniform tensile strength in kg/cm^2

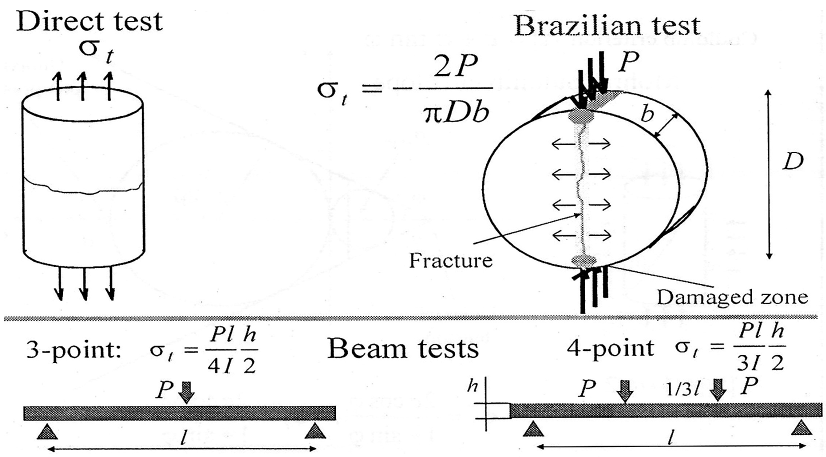


__Figure 2.6:__ Brazilian Test for Tensile Strength of Rock

b) *Flexural Strength or Bending Test (Modulus of Rupture)*

The flexural strength or modulus of rupture is a measure of the outer fibre tensile strength of a material. This can be determined by loading a cylindrical specimen in a three point loading device to fracture (Refer Figure 2.7)

$$R_o = \frac{8F_cL}{\pi D^3}$$

R_0 is flexural strength of rock in kg/cm^2

F_c is the applied compressive load at failure in kg

L is length between the bearing edges of the lower plate in cm

For a prismatic specimen of thickness d and width b

$$R_o = \frac{8F_cL}{2bd^2}$$

__Figure 2.7__: Flexural Strength Test (Modulus of Rupture)

2.7 UNCONFINED SHEAR STRENGTH

Indirect shear test is properly known as punch shear test. Usually, shear test are single shear test, double shear test, punch shear test, torsion shear test [Refer Figures 2.8 (a), 2.8 (b) & 2.9 (a), 2.9 (b)].

Usually, the measured shear strength is not inversely proportional to the cross-sectional area.

1. For single shear test, the shear strength S_o *is*

$$S_o = \frac{F_c}{A}$$

Where F_c is the force in the direction of the plane *'A'* necessary to cause failure.

A = cross-sectional area of specimen

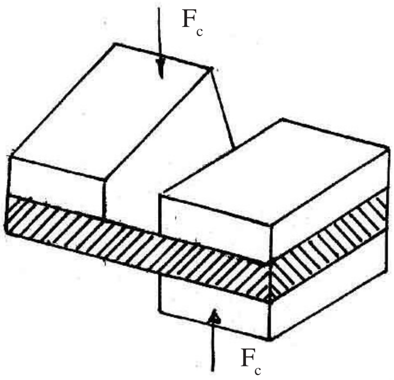


Figure 2.8 (a): *Single Shear Test*

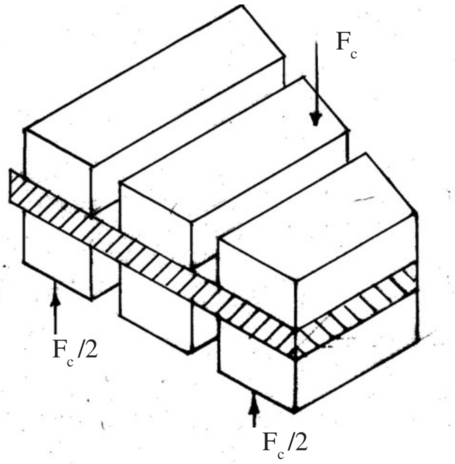


Figure 2.8 (b): *Double Shear Test*

2. For Double Shear test, the shear strength S_o *is*

$$S_o = \frac{F_c}{2A}$$

3. For Punch Shear test, the shear strength S_o *is*

$$S_o = \frac{F_c}{2\pi ra}$$

where, a = thickness of the specimen and
r = radius of punch

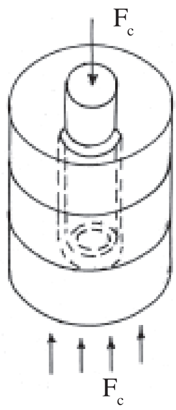


Figure 2.9 (a): *Punch Shear Test*

4. For Torsional Shear test, the shear strength S_o *is*

$$S_o = \frac{16\, M_c}{\pi D^3}$$

where, M_c = applied torque at the failure
D = Diameter of the cylinder

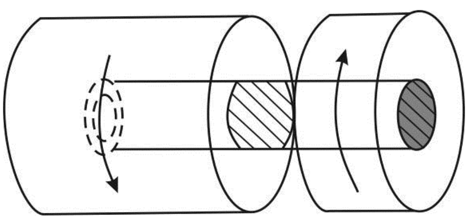


Figure 2.9 (b): *Torsional Shear Test*

2.8 TRIAXIAL COMPRESSIVE AND SHEAR STRENGTH

The triaxial compressive and shear strength is required for calculating the bearing capacity of foundation rock for surface structure and in determining the strength of mine pillar and other parts of underground structure (Refer Figure 2.10).

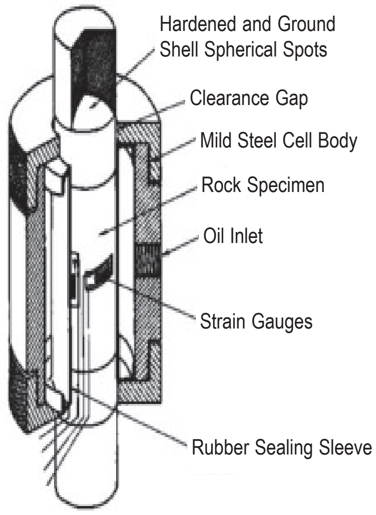


Figure 2.10: *Triaxial Compressive and Shear Strength*

In triaxial test, a constant hydraulic pressure $-p$ may be applied on the curved surface of cylindrical specimen together with applying compressive axial load to the end of specimen till the specimen fails. To prevent the penetration of hydraulic fluid into the pore space in specimen, rubber jacket may be provided to specimen. If F_c is axial load at failure, the principal stresses in specimen at the failure are :

$$\sigma_3 = \frac{F_c}{A} \quad \text{and} \quad \sigma_1 = -p$$

where, A is the end area of the specimen.

To determine the triaxial compressive strength of a given material, usually six specimens are tested under the triaxial machine. The resulting different value of radial pressure σ_1 & σ_3 gives the functional relationship $\sigma_3 = f(\sigma_1)$. For each σ_1 and σ_3 values, Mohr's circle can be constructed. Envelop curves tangent to the circle $\tau_0 = f(\sigma_0)$. The intercept of the envelop with the τ axis is the triaxial shear strength S_0 of the material (Refer Figure 2.11)

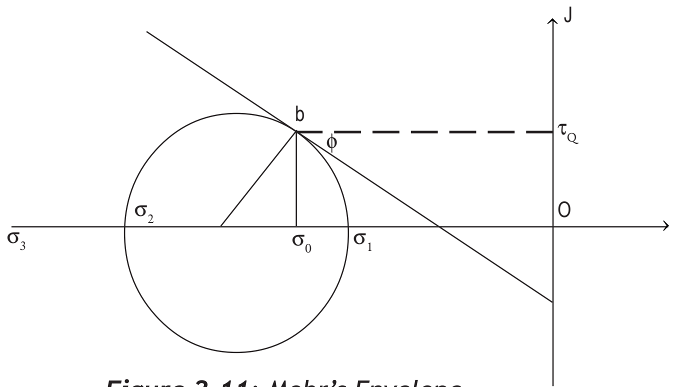


Figure 2.11: *Mohr's Envelope*

Mohr's Circle

ϕ = the angle internal friction formed by tangent to the circle at p and the direction of the τ axis.

Q = Angle of failure plane with respect to axial direction

μ = the coefficient of internal friction can be calculated from tan2Q = 1/μ

σ_1 and σ_3 relationship is approximately linear for many rocks.

$\tau_Q = +(S_o - \sigma_Q \tan\phi)$

σ_Q and τ_Q are the normal and shear stresses acting on the failure plane.

S_o is the shear strength of the rock.

tanφ is the slope of the envelope curves.

A decrease in modulus of elasticity with increasing tensile stress is the characteristics of most rocks.

2.8.1 Effect of the Pore Pressure

The material in triaxial stress is subjected to a pore pressure p_0 and principal stresses σ_3 and ($\sigma_2 = \sigma_1$). The pore pressure reduces the principal stresses so that effective principal stresses σ_1, and σ_3, are

$$\sigma_3, = \sigma_3 - p_0$$
$$\sigma_1, = \sigma_1 - p_0$$

Stress difference ($\sigma_3 - \sigma_1$) and slope of the envelope remain unchanged.

$$\tau_Q = So - (\sigma_Q - p_Q) \tan\phi$$

Presumably the drainage through joints, fractures, faults, parting and watercourses are such that water pressure gradient may be increased slowly with the distance from the surface of openings, i.e., tunnel, adit, shaft, single openings may have larger gradient than the extensive mined area. Thus the most likely condition in rock surrounding an underground opening is a moist or nearly saturated rock with a comparatively small pressure gradient developing behind rock surface.

2.8.2 Void Ratio

Void ratio, in materials science, is related to porosity and defined as the ratio between

$$e = \frac{V_V}{V_S} = \frac{V_V}{V_T - V_V} = \frac{\phi}{1 - \phi} \quad \text{and} \quad \phi = \frac{V_V}{V_T} = \frac{V_V}{V_S + V_V} = \frac{e}{1 + e}$$

where e is void ratio, ϕ is porosity, V_V is the volume of void-space (such as fluids), V_S is the volume of solids and V_T is the total or bulk volume. This figure is relevant in composites, in mining (particular with regard to the properties of tailings) and in soil science. In geotechnical engineering, it is considered as one of the variables of soil's state and is represented by the symbol *'e'*.

2.8.3 Bulk Density

Bulk density is a property of powders, granules and other "divided" solids, especially used in reference to mineral components (soil, gravel), chemical substances (pharmaceutical) ingredients, foodstuff or any other masses of corpuscular or particulate matter. It is defined as the mass of many particles of the material divided by the total volume they occupy. The total volume includes particle volume, inter-particle void volume and internal pore volume.

Bulk density is not an intrinsic property of a material. It can change depending on how the material is handled. For example, a powder poured into a cylinder will have a particular bulk density, if the cylinder is disturbed, the powder particles will move and usually settle closer together, resulting in a higher bulk density. For this reason, the bulk density of powders is usually reported both as "freely settled" (or "poured" density) and "tapped" density (where the tapped density refers to the bulk density of the powder after a specified compaction process, usually involving vibration of the container).

2.8.4 Angle of Repose

The *angle of repose* or, more precisely, the *critical angle of repose*, of a granular material is the steepest angle of descent or dip of the slope relative to the horizontal plane when material on the slope face is on the verge of sliding. This angle is given by the number (0°– 90°).

When bulk granular materials are poured onto a horizontal surface, a conical pile will form. The internal angle between the surface of the pile and the horizontal surface is known as the angle of repose and is related to the density, surface area & shapes of the particles and the coefficient of friction of the material. Material with a low angle of repose forms flatter piles than material with a high angle of repose.

The term has a related usage in mechanics, where it refers to the maximum angle at which an object can rest on an inclined plane without sliding down. This angle is equal to the arc-tangent of the coefficient of static friction μ_s between the surfaces.

2.8.5 Porosity or Void Fraction

Porosity or *void fraction* is a measure of the void (i.e., "empty") spaces in a material, and is a fraction of the volume of voids over the total volume, between 0–1, or as a percentage between 0–100 %. The term is used in multiple fields including pharmaceutics, ceramics, metallurgy, materials, manufacturing, earth sciences and construction.

2.8.6 Permeability

Permeability in fluid mechanics and the *earth sciences* (commonly symbolised as κ or k) denotes the measure of the ability of a porous material(such as, a rock or unconsolidated material) to allow fluids to permeate through it.

2.8.7 Specific Storage

Specific storage (S_s), *storativity (S)*, *specific yield* (S_y) and *specific capacity* are material physical properties that characterise the capacity of an aquifer to release groundwater from storage in response to a decline in hydraulic head. For that reason they are sometimes referred to as "storage properties". In the field of hydrogeology, these properties are often determined using some combination of field hydraulic tests (e.g., aquifer tests) and laboratory tests on aquifer material samples.

The *specific storage* is the amount of water that a portion of an aquifer releases from storage per unit mass or volume of aquifer per unit change in hydraulic head, while remaining fully saturated.

Mass specific storage is the mass of water that an aquifer releases from storage per mass of aquifer per unit decline in hydraulic head:

$$(S_s)_m = \frac{1}{m_a}\frac{dm_w}{dh}$$

where, $(S_s)_m$ is the mass specific storage ($[L^{-1}]$);

m_a is the mass of that portion of the aquifer from which the water is released ([M]);

dm_w is the mass of water released from storage ([M]); and

dh is the decline in hydraulic head ([L]).

2.8.8 Internal Friction and Cohesion

Angle of internal friction for a given soil is the angle in the graph (Mohr's circle) of the shear stresses at which shear failure occurs. Angle of internal friction φ can be determined in the laboratory by Direct Shear test or the Triaxial Stress Test (Refer Figure 2.12). Cohesion is the force that holds together molecules or like particles in their soil. Cohesion 'c' is usually determined in the laboratory using triaxial test or the unconfined compressive strength test.

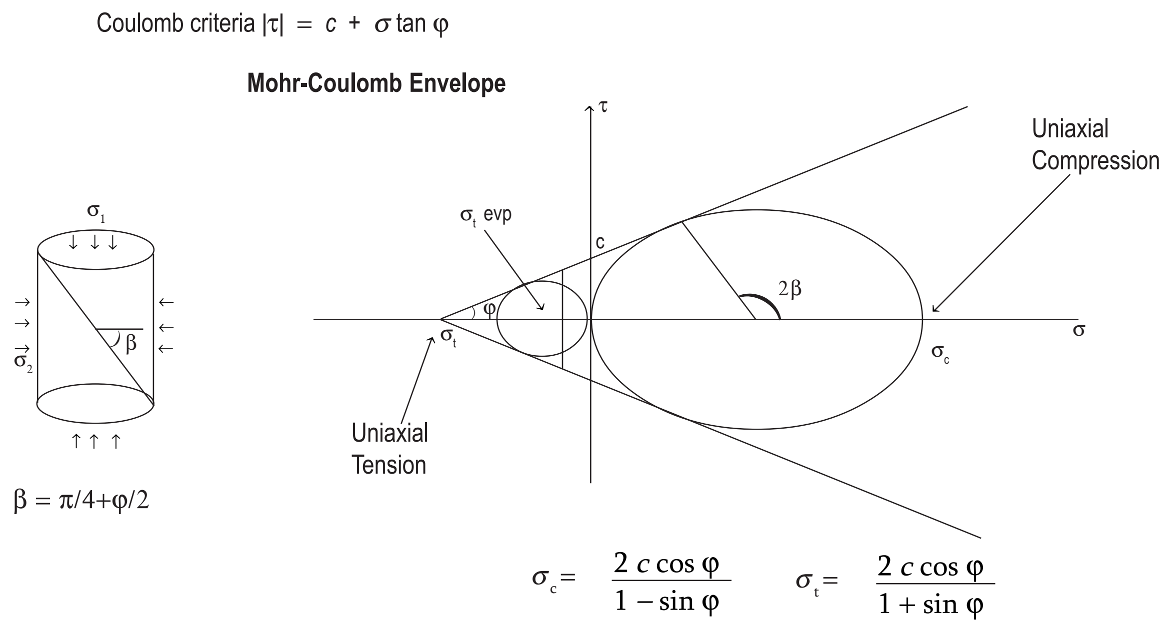


Figure 2.12: Relation Between Friction and Cohesion

2.8.9 Atterberg Limits

The *Atterberg Limits* are a basic measure of the nature of fine-grained soil. Depending on the water content of the soil, it may appear in four states — solid, semi-solid, plastic and liquid. In each state the consistency and behaviour of a soil is different and so are its engineering properties. Thus, the boundary between each state can be defined based on a change in the soil's behaviour. The Atterberg Limits can be used to distinguish between silt and clay, and it can distinguish between different types of silts and clays. These Limits were created by Albert Atterberg, a Swedish chemist. They were later refined by Arthur Casagrande. These distinctions in soil are used in picking the soils to build suitable structures. These tests are mainly used on clayey or silty soils since these are the soils that expand and shrink due to moisture content. Clays and silts react with the water and thus change sizes and have varying shear strengths. Thus these tests are used widely in the preliminary stages of building any structure to insure that the soil will have the correct amount of shear strength and not too much change in volume as it expands and shrinks with different moisture contents.

2.8.10 Shrinkage Limit

The *Shrinkage Limit* (SL) is the water content where further loss of moisture will not result in any more volume reduction. The test to determine the Shrinkage Limit is ASTM International D4943. The Shrinkage Limit is much less commonly used than the liquid and plastic limits.

2.8.11 Plastic Limit

The *Plastic Limit* (PL) is the water content where soil transitions take place between brittle and plastic behaviour. A thread of soil is at its plastic limit when it begins to crumble when rolled to a diameter of 3 mm. To improve test result consistency, a 3 mm diameter rod is often used to gauge the thickness of the thread when conducting the test. The Plastic Limit test is defined by ASTM standard test method D 4318.

2.8.12 Liquid Limit

The *liquid limit* (LL) is the water content at which a soil changes from plastic to liquid behaviour. The original liquid limit test of Atterberg's involved mixing a pat of clay in a round-bottomed porcelain bowl of 10–12 cm diameter. A groove was cut through the pat of clay with a spatula, and the bowl was then struck many times against the palm of one hand. Casagrande, subsequently standardised the apparatus and the procedures to make the measurement more repeatable. Soil is placed into the metal cup portion of the device and a groove is made down its center with a standardised tool of 13.5 millimetres (0.53 in) width. The cup is repeatedly dropped 10 mm onto a hard rubber base during which the groove closes up gradually as a result of the impact. The number of blows for the groove to close is recorded. The moisture content at which it takes 25 drops of the cup to cause the groove to close over a distance of 13.5 millimetres (0.53 in) is defined as the liquid limit. The test is normally run at several moisture contents, and the moisture content which requires 25 blows to close the groove is interpolated from the test results. The Liquid Limit test is defined by ASTM standard test method D 4318. The test method also allows running the test at one moisture content where 20 to 30 blows are required to close the groove; then a correction factor is applied to obtain the liquid limit from the moisture content.

The following is when you should record the N in number of blows needed to close this 1/2-inch gap:

The materials needed to do a Liquid limit test are as follows:

* Casagrande cup (liquid limit device)
* Grooving tool
* Soil pat before test
* Soil pat after test

Another method for measuring the liquid limit is the *fall cone test*. It is based on the measurement of penetration into the soil of a standardised cone of specific mass. Despite the universal prevalence of the Casagrande method, the fall cone test is often considered to be a more consistent alternative because it minimises the possibility of human variations when carrying out the test.

The importance of the liquid limit test is to classify soils. Different soils have varying liquid limits. Also to find the plasticity index of a soil you need to know the liquid limit and the plastic limit.

The values c and 'ϕ' are found by testing a soil sample in the laboratory triaxial or shear box test. Great care must be taken as 'c' and 'ϕ' can vary depending on:

a) The stress range
b) Sample preparation
c) Sample orientation
d) Sample size
e) Rate of testing
f) Softening
g) Progressive failure

The change in 'c' and 'ϕ' as a result of these effect is discussed in Skempton and Hutchinson (1969) state-of-the-art paper on natural slopes and embankment foundation.

2.8.13 Slake Durability Index Test

The test is intended to access the resistance offered by a rock sample to weakening and disintegration when subjected to two standard cycles of drying and wetting.

The *Slake Durability Index* is calculated as the percentage ratio of final to initial dry sample masses as follows:

Slake durability index Id_2 = (B – C)/(A – C)) x 100%.

where, *A* is mass of (drum + sample)

B is the mass of drum plus retained portion of the sample after cooling

C is the mass of drum

Index taken after three or more cycles of slaking and drying may be useful for evaluating rock of higher durability. Slake durability index is used for waste dump, tailing dam design and coal mine dumps or spoils.

2.8.14 California Bearing Ratio

The *California Bearing Ratio* (CBR) is a penetration test for evaluation of the mechanical strength of road subgrades and base curves. It was developed by the California Department of Transportation.

The test is performed by measuring the pressure required to penetrate a soil sample with a plunger of standard area. The measured pressure is then divided by the pressure required to achieve an equal penetration on a standard crushed rock material. The CBR test is described in ASTM Standards D1883-05 (for laboratory-prepared samples) and D4429 (for soils in place in field), and AASHTO T193. The CBR test is fully described in BS 1377: Soils for civil engineering purposes: Part 4, Compaction related tests.

The CBR rating was developed for measuring the load-bearing capacity of soils used for building roads. The CBR can also be used for measuring the load-bearing capacity of unimproved airstrips or for soils under paved airstrips. The harder the surface, the higher the CBR rating. A CBR of 3 equates to tilled farmland, a CBR of 4.75 equates to turf or moist clay, while moist sand may have a CBR of 10. High quality crushed rock has a CBR over 80. The standard material for this test is crushed California limestone which has a value of 100.

$$\text{CBR} = \frac{P}{P_s} \; 100 \text{ CBR} = \text{CBR } [\ \%]$$

P = measured pressure for site soils [N/mm²]

P_s = pressure to achieve equal penetration on standard soil [N/mm²]

2.8.15 Direct Shear Test

A direct shear test also known as shearbox test is a laboratory or field test used by geotechnical engineers to measure the shear strength properties of soil or rock material, or of discontinuities in soil or rock masses. The test is performed on three or four specimens from a relatively undisturbed soil sample. A specimen is placed in a shear box which has two stacked rings to hold the sample; the contact between the two rings is at approximately the mid-height of the sample. A confining stress is applied vertically to the specimen, and the upper ring is pulled laterally until the sample fails, or through a specified strain. The load applied and the strain induced is recorded at frequent intervals to determine a stress-strain curve for the confining stress.

Direct Shear tests can be performed under several conditions. The sample is normally saturated before the test is run, but can be run at the in situ moisture content. The rate of strain can be varied to create a test of undrained or drained conditions, depending whether the strain is applied slowly enough for water in the sample to prevent pore-water pressure buildup.

Several specimens are tested at varying confining stresses to determine the shear strength parameters, the soil cohesion (*c*) and the angle of internal friction (commonly friction angle) (φ). The results of the tests on each specimen are plotted on a graph with the peak (or residual) stress on the x-axis and the confining stress on the y-axis. The y-intercept of the curve which fits the test results is the cohesion, and the slope of the line or curve is the friction angle. The portable shear box is shown in Figure 2.13.

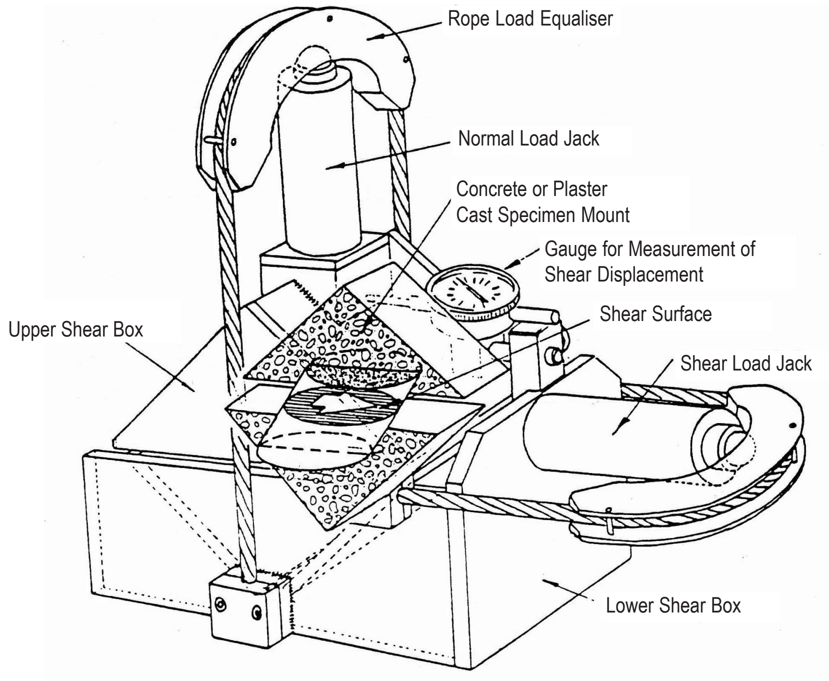


***Figure 2.13**: Portable Shear Box*

2.8.16 Proctor Compaction Test

The Proctor compaction test is a laboratory method of experimentally determining the optimal moisture content at which a given soil type will become most dense and achieve its maximum dry density. The term Proctor is in honour of R.R. Proctor, who in 1933 showed that the dry density of a soil for a given compactive effort depends on the amount of water the soil contains during soil compaction. Later on, his test was updated to create the modified Proctor compaction test.

These laboratory tests generally consist of compacting soil at known moisture content into a cylindrical mould of standard dimensions using a compactive effort of controlled magnitude. The soil is usually compacted into the mould to a certain amount of equal layers, each receiving a number blows from a standard weighted hammer at a specified height. This process is then repeated for various moisture contents and the dry densities are determined for each. The graphical relationship of the dry density to moisture content is then plotted to establish the compaction curve. The maximum dry density is finally obtained from the peak point of the compaction curve and its corresponding moisture content, also known as the optimal moisture content.

The testing described is generally consistent with the American Society for Testing and Materials (ASTM) standards, and are similar to the American Association of State Highway and Transportation Officials (AASHTO) standards. Currently, the procedures and equipment details for the standard Proctor compaction test is designated by ASTM D698 and AASHTO T99. Also, the modified Proctor compaction test is designated by ASTM D1557 and AASHTO T180.

2.9 EFFECT OF GROUNDWATER IN SLOPE ENGINEERING

The incidence of slope failure in working mines during or shortly after periods of intense rainfall indicates the degree to which rainfall and subsequent movement of groundwater affect slope stability. A knowledge of groundwater conditions is needed for the analysis and design of slopes. The groundwater regime is often the only natural parameter that can be economically changed to increase the stability of slopes.

Water affects the stability of slopes in the following ways:

a) By generating pore pressure, both positive and negative, which alter stress conditions;

b) By changing the bulk density of the slope forming material;

c) By both internal and external erosion; and

d) By changing the material constituents of the slope forming material.

2.9.1 Water Balance and the Hydrological Cycle

Water, whether in the solid , liquid or gaseous (vapour) form, is continually in circulation by transformation of its state and by its movement between land, sea and air. This continuous movement is called the hydrological cycle.

In terms of the stability of slopes, the land-based portion of the hydrological cycle is most intense. Inflow to the system arrives as rainfall which can be extremely intense. Water flow from the system can be as run off, evapo-tanspiration, and subsurface underflow. Changes of storage within the system is that part of the rainfall which becomes incorporated into the groundwater system as recharge. The above elements form the basis of the water balance equation.

Rainfall = Evaporation + Runoff + Subsurface underflow + Change in soil moisture + Change in groundwater storage

Changes in groundwater are critical to slope stability as it is these elements in the water balance equation that effectively alter the degree of saturation of the ground above the water table and the elevation of the water table itself.

Runoff is that proportion of rainfall that flows from a catchment into streams, lakes or the sea. It consists of surface runoff and groundwater runoff, where groundwater runoff is derived from rainfall that infiltrates into soil down to the water table and then percolates into stream channels. The amount of runoff in any given catchment depends on variety of factors such as the condition and nature of the soil and bedrock, the intensity and duration of rainfall, the slope angle, the surface of cover and the antecedent conditions within the catchment. The amount or depth of runoff maybe calculated by gauging the flow in streams which drain the catchment. The runoff coefficient or runoff percentage is defined as the proportion of rainfall that flows from a catchment as a percentage of the total depth of rainfall over the catchment areas.

Infiltration is defined as the movement of water from the ground surface into the soil or rock through the pores or interstices of the ground mass (i.e. the absorption of water by the soil). Infiltration can be further divided into that part which contributes to the water content of the unsaturated zone, and the part which recharges the saturated groundwater system. Some recharge to the saturated groundwater system may be lost as groundwater runoff, whilst recharge to the unsaturated zone may be lost by transpiration or evaporation .When an unsaturated zone exists in a soil it is said to have a soil moisture deficit. Recharge to this zone reduces the deficit until the soil becomes fully saturated, at which time the soil moisture deficit is equal to zero.

2.9.2 Types of Groundwater Flow

Water flows through soil or rock in various ways depending on the nature of the ground. Water-transmitting soil or rock units are called aquifers. Different types of aquifer demonstrate different modes of groundwater flow, such as, intergranular, fissure and conduit flow.

Intergranular flow is groundwater flow between the individual component grains that make up a soil or rock. This type of flow must closely follow the Darcy Concept of flow through a homogeneous isotropic medium of uniform grain size. In practice, however most water bearing strata exhibit intergranular or homogenous flow and path-preferential flow through fissures or conduits within the stratum. Joints within a soil/rock mass can have a significant effect on groundwater levels and hence contribute towards slope in stability. An aquifer, therefore, can be simply defined as a permeable water bearing stratum that transmits water under normal head or hydraulic gradient. An aquiclude is a stratum that may contain pore water but is not permeable enough to transmit water even under considerable hydraulic head. The term acquitter is used to indicate a stratum that shows limited water-transmitting capabilities.

Groundwater in aquifers not only exhibits intergranular and path-preferential flow characteristics related to the granular or fissured structure of the aquifer unit, but also *exhibits confined or unconfined* flow characteristics. In the former, groundwater is normally confined at the top by an impermeable stratum (aquiclude). The aquifer unit is therefore gets fully saturated, and the piezometric (hydraulic) head is above the lower boundary of the confining medium. In unconfined flow situations, ground water does not fully occupy the potential aquifer, and a free water surface (water table) exists within the aquifer. Water table in aquifers can either be main table or *perched water table.* The main water table in any aquifer is the surface of the zone of complete saturation where water flows laterally under gravity, in the ground surface. Above the main water table, the soil or rock infiltration occurs through the soil or rock above the phreatic surface, and usually discharges to streams or the sea.

Perched water table exist above the main water table where a localised reduction in basal permeability occurs in conjunction with recharge from above. Perched water tables may be transient, developing rapidly in response to heavy rainfall and dissipating quickly or permanently, responding to seasonal variations in rainfall level.

2.9.3 Pore Pressures and Effective Stress

2.9.3.1 Pore Water in Soils

a) Water Under Static/Equilibrium Conditions: Water obeys laws of hydraulics and when at rest exerts a pressure equivalent to the height/depth of water above any point. Water table is the level at which pressure of water is atmospheric, i.e., pore water pressure = 0. Usually, water level tends to follow ground profile although at a slightly flatter angle. Below the water table, the water pressure increases with increasing depth, with positive pore pressure. Above the water table water may still occur in substantial quantities and is maintained in place by capillary attraction and surface tension negative pore pressure. This produces a soil suction, which increases the apparent cohesion. It is more in fine soils as compared to coarse-grained materials.

A portion of capillary water exists as a very thin film of water surrounding individual grains and is called Adsorbed water. Which tends to have different properties compared to ordinary water including, greater viscosity, higher boiling point and cannot be removed by air drying at ordinary temperatures. The amount of adsorbed water is usually considered as the difference in weight of sample dried at 105–110 °C and in air.

Subsurface water may be divided into zones of positive and negative pore pressure. The dividing line is the groundwater table where the pressure is equal to atmospheric pressure. The groundwater table or the phreatic surface is generally determined from the level of water in an open standpipe. Atmospheric pressure is usually taken as the zero pressure datum and so a positive pore pressure zone exists below the water table. If there is no groundwater flow, the pore pressure is hydrostatic and the water level measured by a piezometer at any depth within the positive pore pressure zone will coincide with the water table.

The negative pore pressure zone occurs above the water table, and the pressure in this zone is less than atmospheric pressure. Water is retained in the soil mass above the water table by capillary action. Immediately above the water table, the soil is saturated up to the level of the saturation capillary head (i.e. the zone of continuous saturation). In this zone, the negative pore pressure is hydrostatic in the no flow situation and variation with depth is linear. Above this zone, the soil is partially saturated and the negative pore pressure does not necessarily vary linearly with depth. The uppermost zone in contact with air is the hygroscopic zone where pore air is continuous and at atmospheric pressure. The height of saturation capillary in soils (which may govern the magnitude of the largest negative pore pressure) relates largely to grain size. With the exception of clays, the finer the soil particles, the larger the saturation capillary head and the higher the negative pore pressure.

2.9.3.2 Effective stresses

The total pressure on a horizontal area within a soil mass is made up of two components, which is transmitted by the solid particles, and that transmitted by the water in voids. When the soil is stressed, as for example by cutting a slope, only the intergranular contacts induce any resistance to deformation and failure. The water having zero shear resistance, is ineffective or neutral and the pore water pressure u is called the neutral pressure. The intergranular pressure is known as the effective pressure and it is "effective" in resisting deformation and volume changes. The effective pressure σ' is therefore defined as the difference between the total external pressure σ and the neutral pressure u.

$$\sigma' = \sigma - u$$

If we consider unit areas then these terms may be defined as in terms of stress.

The importance of this is that volume changes and deformations depend on changes in effective stresses.

$$\frac{\Delta V}{V} = \zeta\,(\Delta\sigma - \Delta\upsilon) = -\zeta\,(\Delta\sigma')$$

where, $\frac{\Delta V}{V}$ = change in volume per unit volume of soil

$\Delta\sigma$ = change in total stress

$\Delta\upsilon$ = change in pore pressure

ζ = compressibility of the soil skeleton

This equation can be shown to be valid whatever be the contact area between particles.

This equation has two significant implications:

a) Long term settlement of structures founded on clay due to excess pore pressures set up during construction and dissipating at a slow rate, causing changes in pore pressure and therefore changes in $\Delta V/V$.

b) Additional settlement due to lowering of groundwater.

Effective stresses are also significant in terms of shear strength, which is largely determined by frictional forces during slip along soil grain contacts — this being a function of the normal stress carried by the soil.

$$\tau' = c' + \sigma_n' \tan \phi'$$

where, τ' = maximum shear resistance on any place

c' = apparent cohesion

σ_n' = $(\sigma_n - u)$

$\tan \phi$ = effective friction angle

For slope stability analysis, it is therefore needed to determine values of c', tan ϕ' and σ_n'. It is relatively easy to get a value for the total stress σ_n but it is quite difficult to obtain values of *u*, which is an independent variable.

• • •

Chapter

3 Site Investigation

3.1 PRINCIPLES OF SITE INVESTIGATION

Site investigation should be regarded as an integral of the mine design process and should not be viewed as an exercise only to be carried out after problems have occurred. The basic of this approach is the information of an engieering model. Although this could be physical model more usually it is a numerical one, describing accurately the mine site conditions. This model may then the "tested" to determine the response to the various engineering problems, e.g. excavation of slopes, construction of foundations, dewatering, etc. The value of any model is determined by its accuracy and representativeness of field conditions, and is thus very dependent on both the quality and quantity of available site investigation data.

3.1.1 Aims of Site Investigation

The basic objectives of any Site investigation are:

a) to assess the suitability of a site for the proposed mine development;

b) to enable the preparation of an adequate and economic design;

c) to foresee and provide against geo-technicical problems during and after operation; and

d) to investigate any subsequent changes in conditions, or the possibility of any failures during operation.

However, underlying these aims is the restraint that the cost of site investigation must be minimised and which in practice, tends to be less than one per cent of the total project cost for civil projects and is significantly less for mining operations.

3.1.2 Organisation of Site Investigations

The usual pattern of any soils-site investigation comprises office and field studies as follows:

Phase I – Preliminary Investigation which involve desk study of available literature, maps, reports and other data including satellite imageries, aerial photographic analysis on a regional scale.

Phase II – Field Investigation which include drilling and boring, excavation of trial pits, penetration testing, sampling, geophysical surveys and measurements of the groundwater regime.

Phase III – Laboratory Testing and evolution of basic geological data.

Phase IV – Field Testing that include in situ tests prior to and during excavation and performance tests on selected or critical structures/areas of the mine.

The data obtained should be compiled into an engineering model and may be best represented by geological cross-sections through the mine, site plans, three-dimensional projections and a variety of other techniques. One real problem with large site investigations is how to handle the large volume of data generated and perhaps more importantly, how to reduce this data to sensible engineering parameters.

All too often there is a gulf between geologists and engineers, with geologists recording the in homogeneity of a deposit and engineers requiring single design parameters. Clearly, careful evolution of site investigation data is essential and any engineering model must firstly be representative of the actual soil conditions and secondly, be manageable in terms of engineering analysis. The value of data banks and case histories cannot be over emphasised here.

One particular problem of site investigations for mining operations is their cost and a clear understanding of the value of investigations, the range and local availability of particular techniques should be additionally understood.

3.2 REVIEW OF SITE INVESTIGATION TECHNIQUES

Site investigation techniques may be considered in two categories—Direct and Remote investigations. Both these categories can contribute significant data to the engineering model and often are complementary to each other. The following features show the context of various standard site investigation techniques:

Geological Aspects

- compilation of existing data
- aerial photograph and satellite imagery analysis
- field mapping and visual inspection
- microscopic analysis of soil samples
- sample description

Field Investigations

- rotary borehole drilling
- trail pit excavations
- sampling (disturbed and undisturbed)
- groundwater measurement (including automatic readouts)
- cone penetration test
- geophysical surveys — seismic
- electrical
- magnetic
- Standard penetration test
- dynamic probing (mackintosh probe)

3.2.1 Field Studies—Direct Investigation Methods

Visual Inspection

One of the easiest and cheapest ways to investigate a site is a "walk over survey" by an Engineer/ Geologist. The role of visual inspections, particularly in low cost investigations, should not be underestimated.

The inspection should be carried out by an experienced engineer or engineering geologist and field data recorded. There are two recommended formats for recording field-data, either in sketch engineering geological maps or on standard report forms.

The standard format allows the recording of basic field-data such as slope angles, slope heights, location of groundwater seepages, material type, size & location of minor slips and perhaps more importantly, the recording of change. Within a mine, changing faces, levels of extraction and field boundaries are the features that make site investigations and slope design in mines very different to normal civil engineering practice.

Boring and Drilling

The boring and drilling of boreholes for site investigation is very common practice and usually involves the production of small diameter holes penetrating through the geological formation. Boreholes allow direct access to the ground and samples (either distributed or undistributed) to be obtained.

Boring Methods tend only to be suitable for soft soils and rocks and result in disturbed samples. However, cable percussion and rotary percussive boring are popular techniques. Holes are sunk by using a clay cutter in dry cohesive soils or a shell in granular strata or below the water table. Casing can be driven down to support the side wall of the borehole. The disturbed samples produced are suitable for identification of the strata and for classification tests. If more sophisticated tests are carried out, the extent of sample disturbance on measured parameters should be assessed.

Rotary Drilling for the recovery of cored samples is probably the commonest form of site investigation. Holes are drilled either vertically or with an inclination up to 45^{o} and a wide variety of rigs and ancillary equipment are used for this purpose. Rotary drilling is usually carried out with a flushing medium which can be water, air, air-foam or mud. The flushing medium has several uses, including the removal of chips of broken material, lubrication and provision of support to the borehole walls.

It is important to select the correct type of core barrel for sampling as adoption of the wrong type can cause disturbance or damage to cores.

a) A single-tube core barrel rotates against the core which is not protected from the drilling fluid—core recovery is seldom satisfactory and it should not be used for site investigation.

b) A double-tube core barrel has an inner tube mounted on bearings so that it does not revolve with the drill strings—it is normally used at sites that have fresh to moderately weathered rocks.

c) Triple-tube core barrels may be used where other methods have been found ineffective and good core recovery is required. Triple-tube barrels have detachable liners within an inner barrel that pitiably protect the core from drilling fluid and from damage during extrusion and subsequent transit.

Records of drilling and boring behavior are an important source of information about site conditions and drillers should be trained to record factual data about:

- The rate of drilling,
- The nature of flushing medium, colour consistency, nature of fragments,
- Loss or percentage return of flushing fluid during drilling,
- Groundwater levels at start and end of drilling
- Equipment used

The major advantage of drilling and boring is that in situ tests can be carried out in the borehole. Standard tests include field permeability testing (using raising, falling or constant head methods), and down hole geophysics. In addition, during drilling, standard penetration tests (SPT) are a useful guide to assess ground consistency. More specialised tests for soil deformation may also be carried out.

The main disadvantage of site investigation with boreholes is that the volume of ground investigated is essentially very small compared to the total site. As boreholes samples are regarded as “point samples” in plan and are vertically drawn, they do not always give a continuous profile. Vertical boreholes are relatively easy to drill, and whilst they are ideal for horizontal strata, the use of inclined boreholes in dipping strata can also be useful. The extrapolation of data from boreholes is therefore a difficult and skilled operation, requiring careful interpretation and a knowledge of the limitation of the technique used. The use of Trail Excavations and Larger Field Excavations particularly in operating mines often allows the persistence of geological features to be better assessed. However, trail excavations tend to be limited in depth, particularly in soft ground, and at times will require installation of temporary support. Existing mine faces are an excellent source of information and borehole data should always be correlated against field exposures.

3.2.2 Groundwater Measurements

Groundwater levels, and more specifically pore pressure measurements, are important measurements taken in the field, having a significant influence on the engineering behaviour of soils, slopes and related excavations.

The simplest measurements are in cased boreholes where the water level gives a direct indication of the groundwater pressure at the base of the casing. For long term applications, piezometers are installed in specified sealed horizons. In granular deposits an open standpipe is usually sufficient but in clays and low permeability deposits closed system devices are required.

3.2.3 Sampling

The drilling of a borehole is really only the first stage in a site investigation and the production of samples for testing is an important aspect of this technique.

Continuous sampling is the best method of defining the range and thickness of materials present. However, interval sampling is more usually carried out, but significant horizons can be missed, especially pre-existing slip surfaces.

When designing site investigations, it is important to specify the size of samples required in relation to the nature of geotechnical laboratory tests that are to be carried out. There is no fixed size requirement as the nature of the ground influences the size of sample required. In fine soils, smaller samples will be required as compared to coarse granular materials.

The type of sample is another critical factor for deciding on the reliability of test data. Samples may be defined as undisturbed or disturbed. The degree of disturbance depends to a large extent on the quality of the drilling equipment, the nature of the ground and the experience of the drill crew. Even so-called undisturbed samples can show signs of significant disturbance such as distortion of layers at sample edge and compression of soft horizons. This disturbance does not preclude laboratory testing if results are assessed against the knowledge of the extent of sample disruption.

It should also be borne in mind that the behaviour of a soil mass is often dictated by the presence of corestones of less weathered material, weaknesses and discontinuities—the very things that tend to result in poor or no samples. Thus it is possible to obtain an intact sample of the soil material which may be unrepresentative of the entire soil mass.

Once the sample has been obtained, it is important that it is retained and protected in the state that it was recovered from the ground. Of particular significance could be the in situ water content and soil structure and it is common practice to wrap samples in aluminium foil and to coat them with a protective layer(s) of sealing wax.

Correct labelling of samples with their borehole designation, depth, inclination, is also important and if not carried out, can result in valueless samples and meaningless data.

3.2.4 Standard Penetration Test (SPT)

This test is most commonly used to give a rough relative measure of the density of granular soils. The procedure is described in BS 1377 (1975) and involves driving a tool with specified standard dimensions into the ground with a standard hammer (usually 63.3 kg). The results can be significantly affected by the testing technique, so while carrying out the test and interpreting the results, the following points should be noted:

a) The borehole casing should be ahead of the borehole, and water balance should be maintained if carrying out the test below the water table.

b) Large diameter rods (BW or equivalent) or smaller rods with rod supports should be used to reduce energy dissipation.

c) An automatic trip hammer should be used to drive the sampler, the accuracy of a monkey and slip wrench is too dependent on the skill of the operator.

The 'N' value is defined as the number of blows required to drive a standard split spoon sampler to a distance of 300 mm. The sampler is initially driven 150 mm to penetrate the disturbed material at the bottom of the borehole before the test is carried out. The operator, having noted the number of blows required for each 75 mm advance of the seating, then notes the number of blows required for each 75 mm advance of the test drive.

It should be noted that the empirical relationship developed for transported soils between N value and foundation design indices, relative density and shear strength are not valid for weathered rocks and residual soils. Corestones, for example, can give misleadingly high values that are unrepresentative of the mass.

Further problems may also occur in thinly bedded materials where it is not possible to sample 300 mm of the same material. In cohesive soils, the test is less reliable than in granular materials due to the development of high pore pressures around the tip of the instrument. These problems are partially overcome by providing a continuous test as discussed in the next Section.

3.2.5 Cone Penetration Testing (CPT)

This technique is one that has found much success in soft ground conditions, for both non-cohesive and cohesive soils.

The principal reasons for this success reflect the accuracy and detailed nature of CPT data and the relative speed and economy with which tests can be carried out, compared to conventional drilling and boring. Another major advantage is that a continuous ground profile can be obtained, which, when correlated with borehole data, can be used to compile a more comprehensive site model. The more accurate knowledge of soil layering so obtained is essential to stability studies where thin layers that might otherwise be missed, can influence slope behaviour significantly.

The principle of performing a cone test is essentially very simple—a standardised probe, with a tip area of 10 cm^2, is pushed by hydraulic jacks into the ground at a constant rate of penetration, and the tip and side resistance measured. The ratio of these two parameters is used as an indicator of the soil type, and empirical correlations established over many years allow direct calculation of engineering parameters, such as, the undrained shear strength of cohesive soils.

There are two main cone types—*mechanical cones*, where penetration resistance is measured by pressure in the hydraulic jacks and *electric cones*, where direct readings on the tip are measured by means of strain gauges.

To enable testing to be carried out in a variety of soils (and soft rocks), additional tools and techniques have been developed. These include:

a) Pore water pressure measurements (piezocone)
b) In situ permeability measurements using the piezocone
c) Soil temperature measurements
d) Inclusion of an inclinometer to check the verticality of the probe hole
e) Resistivity measurements to determine the in situ density of sand deposits.

3.2.6 Dynamic Probing

The Mackintosh Probe is a handprobe which can be used down to depths of 10–15 m in soft ground. The probe point is 3 mm in diameter. Blow counts for each 100 mm penetration are recorded and plotted. For investigation of an existing slope, a large number of probes are put down initially to obtain a general indication of the subsurface profile. This information is used to assist in the location of subsequent trial pits and drillholes. The probe can also be used for indicating the state of compaction of buried fill and the thickness of fill layers. A weight correction should be applied for depths greater than 5 m.

3.2.7 Remote Techniques

These techniques are essentially geophysical and depend on subjecting the soil to some form of disturbance and measuring the ground response. These “indirect” tests have the advantage that a larger volume of ground may be tested, but suffer from a number of limiting criteria. The interpretation of geophysical data is very specialised which relys on the assessment of differences of measurable properties, to derive by inference the changes in substratum conditions. Interpretation is best attempted with correlatory boreholes or CPT profiles and by an experienced geophysicist. The emphasis in recent years on large scale geophysical surveys for oil exploration has resulted in an abundance of geophysical correlations and data, which have not always been fully utilised in the civil and mining engineering industries.

3.2.8 Seismic Testing

The use of seismic refraction tests is well-established in site investigation practice. The technique depends on the assumption that seismic waves have a characteristic velocity dependent on the density and elastic properties for different materials. Seismic refraction will only work for layers showing increasing density with depth, and seismic waves are refracted across the boundary.

Waves are generated by either a percussion (hammer) source or an explosive source and comprise a number of wave types. Compression P-waves are the commonest, and more significantly, the easiest to recognise. However, beneath the water table they travel with the characteristic velocity of the groundwater, resulting in less accurate measurements. Shear S-waves are particularly useful and are sensitive to variations in the geological profile. Direct correlations with the shear modulus of the soil are available and may be used to assess the degree of compaction of granular deposits.

The seismic refraction method relies upon the earliest time of arrival of seismic waves, refracted at the interface between different strata, at a chain of geophones. These times are plotted against distance from the seismic source, and a series of straight lines of varying gradient are thus obtained. The gradients of the lines are used to determine the depth of each interface. Problems can arise when weak layers are overlain by stronger material or when layers exist that are thin relative to the overall depth being surveyed. Control boreholes can help to overcome some of these problems and a check should be made to establish whether a seismic method can be used.

3.2.9 Electrical Methods

Of the various electrical methods of surveying (resistivity, self potential and induction) the resistivity technique is the most popular and commonly used. The ground resistivity is a direct function of the composition of the soil layers and any contained fluid (electrolyte). Changes in apparent resistivity are determined by generating an electric field between two fixed points. Variations in the apparent resistivity can be calculated and correlated with layer changes.

3.2.10 Summary

The economic aim of a site investigation is to obtain the maximum amount of good quality data AT minimum cost. Mine operators must be familiar with the necessity for site investigation and the range of techniques available in the region. Different techniques should be combined to give a range of results that can be used as the basis for defining the site conditions. The limitations of each method are important and the control on output of the quality (i.e., accuracy, reliability, repeatability and representativeness) of the input data needs careful consideration.

3.3 LABORATORY AND FIELD TECHNIQUES FOR ROCK MASS CHARACTERISATION

Whereas the ground investigations set up for the design of a highway cutting will be directly aimed towards collection of relevant geological and geotechnical data, the open pit designer is seldom so fortunate, and must often glean his design parameters from investigations more aimed towards orebody proving and overburden assessments. But having accepted this, the collection of data is generally carried out using the following techniques.

3.3.1 Fracture Mapping

Regional Geology

Assessment made from overall large-scale geological features derived from aerial photographs, satellite imagery and topographical maps. Also detailed regional geological maps, and useful adjacent exposures, such as mines, quarries, cuttings, river beds.

Structural Mapping

Slow and often tedious, but vitally important to the investigation. Tool used principally is the geological compass, such as that made by Breithaupt or Zeiss. All structural mapping is subject to bias unless carried out on three mutually perpendicular planes, a situation rarely possible in slope development.

Photogrammetric Structural Mapping

Subject to at least the same limitations as above, but does enable mapping of otherwise inaccessible areas.

These three techniques serve to define the large-scale orientations, spacing and interrelation of major structural features. Of considerable importance is the roughness of these joints, and this will later be shown in its effect on the frictional resistance to shear of the joint.

The foregoing has dealt only with surface measurement – the even more important aspects of subsurface definition are dealt with using diamond core drilling. Here, too, there is often a conflict between the contract and scientific requirements – the core must be drilled carefully (= slowly) and recovered gently. Double tube or triple tube core barrels are essential to recover good quality core, and a split inner tube is desirable. Where good quality core is obtained, orientation devices can be used to enable true orientation of the intersected structural features to be made.

3.3.2 Stereonets

A rock mass consists of blocks of intact material separated by discontinuities. In some rock masses, these discontinuities tend to form in-sets or families—in others, the discontinuities tend to be more random in their orientation. Use of devices, such as, stereonets greatly aids the definition of sets (or the proof of randomness).

Any plane can be uniquely defined in space by the use of two parameters—dip, which is the maximum inclination of the plane, and *dip direction* or *dip asimuth,* which is the bearing of the horizontal trace of the dip, measured clockwise from North. This form of definition is preferred to that involving strike, as only two numeric items provide the unique definition. There are two principal forms of projection—the equal area (or Lambert) projection is the one that is predominantly used.

Figure 3.1 below shows how the plane and sphere relate —not only is the plane characterised by a great circle trace on the surface of the hemisphere, but also by the pole of that plane, which can be regarded as the intersection of the planes' unit vector with the hemisphere surface.

The Figure on the right shows the mechanics of projecting any point on the hemisphere's surface onto a circular plane. The following three examples are reproduced directly from Rock Slope Engineering.

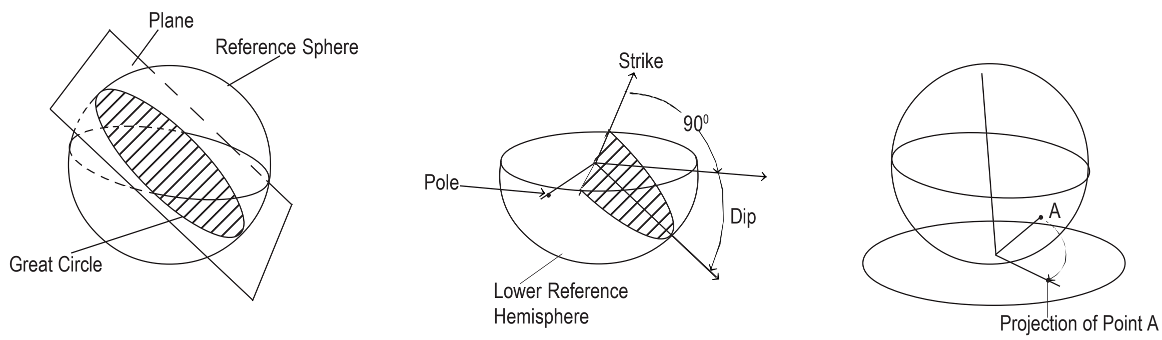
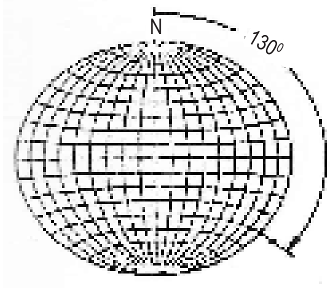


Figure 3.1: *Plotting of Stereonets*

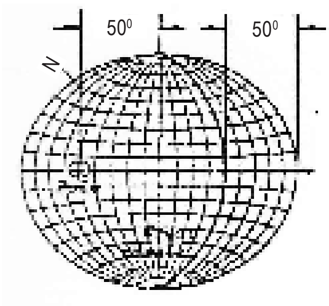
To analyse a set of orientation readings they first require plotting on the net. This initial plotting may show some clustering if the mapping is that of a consistent and well-ordered rock mass, but probably a random element will tend to obscure the joint sets, and it is quite likely that a result such as this shown in the figure below, left, will be obtained. Only by use of statistical contouring will a clearer pattern emerge, such as that on the right refer Figure 3.2.



Construction of a great circle and a pole representing a plane

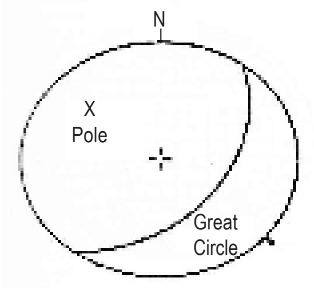
Consider a plane dipping at 50^0 in a dip direction of 130^0. The great circle and the pole representing this plane are constructed as follows:

Step 1**:** With the tracing paper located over the stereonet by means of the centre pin, trace the circumference of the net and mark the north point. Measure off the dip direction of 130^0 clockwise from north and mark this position on the circumference of the net.

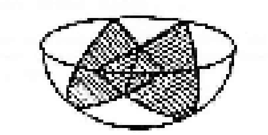


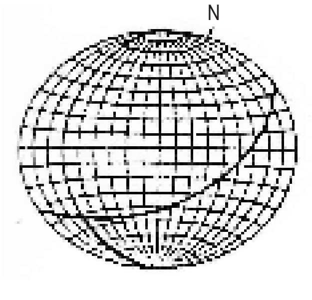
Step 2**:** Rotate the tracing about the centre pin until the dip direction mark lies on the W-E axis of the net, i.e., the tracing is rotated through 40^0. Measure 50^0 from the outer circle of the net and trace the great circle which corresponds to a plane dipping at this angle.

The position of the pole, which has a dip of (90^0–50^0), is found by measuring 50^0 from the centre of the net as shown or, alternatively, 40^0 from the outside of the net. The pole lies on the projection of the dip direction line which, at this stage in the construction, is coincident with W-E axis of the net.



Step 3**:** The tracing is now rotated back to its original position so that the north mark on the tracing coincides with the north mark on the net. The final appearance of the great circle and the pole representing a plane dipping at 50^0 in a dip direction of 130^0 is as illustrated.

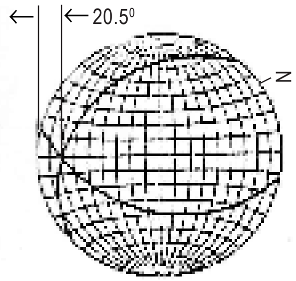




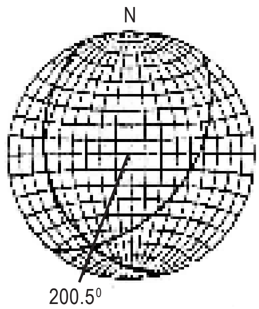
Determination of the line of intersection of two planes

Two planes, having dips of 50^0 and 30^0 and dip directions of 130^0 and 250^0 respectively, intersect. It is required to find the plunge and the trend of the line of intersection.

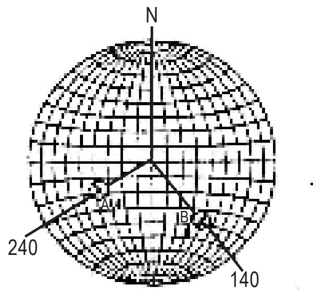
Step1**:** One of these planes has already been described above and the great circle defining the second plane is obtained by marking the 250^0 direction on the circumference of the net, rotating the tracing until this mark lies on the W-E axis and tracing the great circle correspond-ing to a dip of 30^0.



Step 2: The tracing is now rotated until the intersection of the two great circles lies along the W-E axis of the stereonet and the plunge of the line of intersection is measured as 20.5^0.



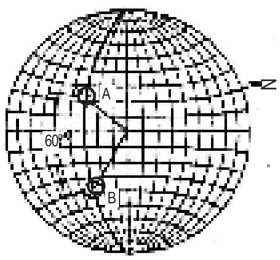
Step 3: The tracing is now rotated until the north mark coincides with the north point on the stereonet and the trend of the line of intersection is found to be 200.5^0.



To determine the angle between two specific lines

Two lines in space, e.g., lines of intersection or normals to planes, are specified by plunges of 54^0 and 40^0 and trends of 240^0 and 140^0 respectively. It is required to find the angle between these lines.

Step 1: The points A and B which defines these lines are marked on the stereonet as described under procedure for locating the pole.



Step 2: The tracing is now rotated until these two points lie on the same great circles on the stereonet and the angle between the lines is determined by counting the small circle divisions between A and B, along the great circles. This angle is found to be 64^0.

The great circle on which A and B lie defines the plane which contains these two lines and the dip and dip directions of this plane are found to be 60^0 and 200^0 respectively.

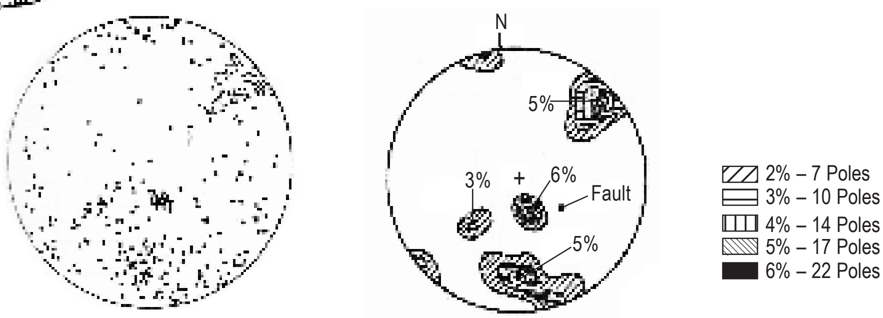


Figure 3.2: *Construction of a Great Circle and a Pole Representing a Plane*

For this course the "floating circle" technique is recommended, and is executed by roving around the net with a circle 1% of net area, counting poles within the circle and plotting this value at the circle centre.

● ● ●

Chapter

4

Rock Mass Classification

4.0 INTRODUCTION

Rock mass classification is useful for empirical design approach that correlates the practical experiences encountered at the previous site to the situation that could be expected at the proposed site. It is widely used in rock engineering. It is used as systematic and practical basis for the design of complex excavations, such as, adit, inclined, stope etc.

It was in 1946 that the Terzaghi rock load classification was introduced for the first time. Since then many modifications took place—Lauffer 1958; Rabceuicz, Pacher & Muller 1964; Deere 1967; Wideham et al. 1972; Bieniaswki 1973; Laubscher 1977; Ghosh & Raju 1981; Kendorski et al. 1983; Gonzales de Vallejo 1983; Unal 1983; Borton et al. 1974; Franklin 1975; International Society of Rock Mechanics 1981.

Rock mass classification is a designed tool. It helps not only to identify the qualitative data – for design, those which influence the behaviour of rock mass and that which enable understanding of the characteristics of each rock mass class – but also enable recommendation of the support guidelines for mines. The advantages of rock mass classification are (i) it gives minimum input data for improving the quality of proposed site; (ii) it provides quantitative information for support system; and (iii) it enables mining professionals in better understanding of the site.

4.1 ENGINEERING ROCK MASS CLASSIFICATION

Rock mass classification schemes have been in the process of evolvement and development for over 100 years. It was Ritter (1879) who attempted to formalise an empirical approach to tunnel design, for the purpose of determining support requirements. While the classification schemes are appropriate for their original application, especially when used within the bounds of the case histories from which they were developed, considerable caution must be exercised in applying rock mass classifications to other rock engineering problems.

Most of the multi-parameter classification schemes [Wickham et al. (1972), Bieniawski (1973, 1989) and Barton et al. (1974)] were developed from civil engineering case histories in which all of the components of the engineering geological character of the rock mass were included. In underground hard rock mining especially at deep levels, rock mass weathering and the influence of water usually are not significant and therefore are ignored. Different classification systems place different emphases on the various parameters, and it is recommended that at least two methods be used at any site during the early stages of a project.

4.1.1 Terzaghi's Rock Mass Classification

The earliest reference to the use of rock mass classification for the design of tunnel support is in a paper by Terzaghi (1946) in which the rock loads, carried by steel sets, are estimated on the basis of a descriptive classification. While no useful purpose would be served by including details of Terzaghi's classification in the context of design of support, it would be, however, interesting to note the rock mass descriptions included in his original paper, because Terzaghi has devoted focus on those characteristics that dominate rock mass behaviour, particularly, in situations where gravity constitutes the dominant driving force.

The clear and concise definitions and the practical comments included in these descriptions are good examples of the type of engineering geology information, which bear useful relevance & significance to the discipline of engineering design.

Terzaghi's descriptions (quoted directly from his paper) are:

- *Intact* rock contains neither joints nor hair cracks. Hence, if it breaks, it breaks across sound rock. On account of the injury to the rock due to blasting, spalls may drop off the roof several hours or days after blasting. This is known as a *spalling* condition. Hard, intact rock may also be encountered in the *popping* condition involving the spontaneous and violent detachment of rock slabs from the sides or roof.
- *Stratified* rock consists of individual strata with little or no resistance against separation along the boundaries between the strata. The strata may or may not be weakened by transverse joints. In such rock the spalling condition is quite common.
- *Moderately jointed* rock contains joints and hair cracks, but the blocks between joints are locally grown together or so intimately interlocked that vertical walls do not require lateral support. In rocks of this type, both spalling and popping conditions may be encountered.
- *Blocky and seamy* rock consists of chemically intact or almost intact rock fragments which are entirely separated from each other and imperfectly interlocked. In such rock, vertical walls may require lateral support.
- *Crushed* but chemically intact rock has the character of crusher run. If most or all of the fragments are as small as fine sand grains and no recementation has taken place, crushed rock below the water table exhibits the properties of a water-bearing sand.
- *Squeezing* rock slowly advances into the tunnel without perceptible volume increase. A prerequisite for squeeze is a high percentage of microscopic and sub-microscopic particles of micaceous minerals or clay minerals with a low swelling capacity.
- S*welling* rock advances into the tunnel chiefly on account of expansion. The capacity to swell seems to be limited to those rocks that contain clay minerals, such as, montmorillonite, with a high swelling capacity.

4.1.2 Classifications Involving Stand-up Time

Lauffer (1958) proposed that the stand-up time for an unsupported span is related to the quality of the rock mass in which the span is excavated. In a tunnel, the unsupported span is defined as the span of the tunnel or the distance between the face and the nearest support, if this is greater than the tunnel span. Lauffer's original classification has since been modified by a number of authors, notably Pacher et al. (1974), which now has become a part of the general tunnelling approach known as the *New Austrian Tunnelling Method*.

The significance of the stand-up time concept is that an increase in the span of the tunnel leads to a significant reduction in the time available for the installation of support. For example, a small pilot tunnel may be successfully constructed with minimal support, while a larger span tunnel in the same rock mass may not be stable without the immediate installation of substantial support.

The New Austrian Tunnelling Method includes a number of techniques for safe tunnelling in rock conditions in which the stand-up time is limited before failure occurs.

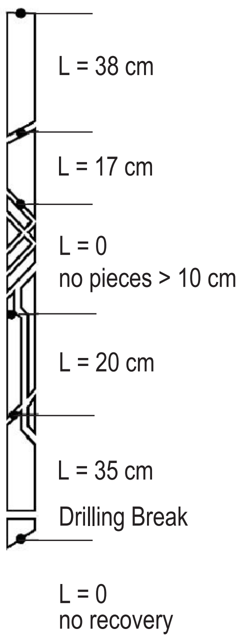
These techniques include the use of smaller headings and benching or the use of multiple drifts to form a reinforced ring inside which the bulk of the tunnel can be excavated.

These techniques are applicable in soft rocks, such as, shales, phyllites and mudstones in which the squeezing and swelling problems, described by Terzaghi (see previous section), are likely to occur. The techniques are also applicable when tunnelling in regions that have excessively broken rocks. Tremendous care, however, need to be observed while attempting to apply these techniques for excavations in hard rocks as different failure mechanisms could occur.

In designing support for hard rock excavations it is prudent to assume that the stability of the rock mass surrounding the excavation is not time-dependent. Hence, if a structurally-defined wedge is exposed in the roof of an excavation, it will fall as soon as the rock supporting it is removed. This can occur at the time of the blast or during the subsequent scaling operation. If it is required to keep such a wedge in place, or to enhance the margin of safety, it is essential that the support be installed as early as possible, preferably before the rock supporting the full wedge is removed. On the other hand, in a highly stressed rock, failure will generally be induced by some change in the stress field surrounding the excavation. The failure may occur gradually and manifest itself as spalling or slabbing or it may occur suddenly in the form of a rock burst. In either case, the support design must take into account the change in the stress field rather than the 'stand-up' time of the excavation.

4.2 ROCK QUALITY DESIGNATION INDEX (RQD)

The Rock Quality Designation Index (*RQD*) was developed by Deere (Deere et al. 1967) to provide a quantitative estimate of rock mass quality from drill core logs. RQD is defined as the percentage of intact core pieces longer than 100 mm (4 inches) in the total length of core. The core should be at least NW size (54.7 mm or 2.15 inches in diameter) and should be drilled with a double-tube core barrel. The correct procedures for measurement of the length of core pieces and the calculation of *RQD* are summarised in Figure 4.1.



Total length of core run = 200 cm

$$RQD = \frac{\Sigma \text{ Length of core pieces} > 10 \text{ cm length}}{\text{Total length of core run}} \times 100$$

$$RQD = \frac{38 + 17 + 20}{200} \times 100 = 55\ \%$$

***Figure 4.1**: Procedure for Measurement and Calculation of* RQD *(After Deere, 1989)*

It is used for core logging and tunnelling. Rock quality designation is related to percentage of core recovery while drilling.

% of Core Recovery	Quality of Rock
25	Very poor
25–50	Poor
50–75	Fair
75–90	Good
90–100	Excellent

The advantage of RQD is that it enables in estimating the support requirement for rock tunnel. The disadvantages, however, are (i) concerns regarding joint orientation, tightness and gauge infilling material (ii) does not provide adequate description of rock mass (iii) it is limited to core quality.

Palmström (1982) suggested that, when no core is available but discontinuity traces are visible in surface exposures or exploration adits, the *RQD* may be estimated from the number of discontinuities per unit volume. The suggested relationship for clay-free rock masses is:

$$RQD = 115 - 3.3\,Jv \qquad (1)$$

where *Jv* is the sum of the number of joints per unit length for all joint (discontinuity) sets known as the volumetric joint count.

RQD is a directionally dependent parameter and its value may change significantly, depending upon the borehole orientation. The use of the volumetric joint count can be quite useful in reducing this directional dependence.

RQD is intended to represent the in situ rock mass quality. When using diamond drill core, care must be taken to ensure that fractures, which have been caused by handling or during the drilling process, are identified and ignored when determining the value of *RQD.*

When using Palmström's relationship for exposure mapping, blast induced fractures should not be included when estimating *Jv.*

Deere's *RQD* was widely used, particularly in North America, after its introduction. Cording and Deere (1972), Merritt (1972) and Deere and Deere (1988) attempted to relate *RQD* to Terzaghi's rock load factors and to rockbolt requirements in tunnels. In the context of this discussion, the most important use of *RQD* is as a component of the *RMR* and *Q* rock mass classifications covered later in this Chapter.

4.2.1 Rock Structure Rating (RSR)

Wickham et al. (1972) expounded on a quantitative method for describing the quality of a rock mass and laid emphasis on selecting appropriate support that is based on Rock Structure Rating (*RSR*) classification. Most of the case histories, used in the development of this system, were for relatively small tunnels supported by means of steel sets, although historically this system was the first to make reference to shotcrete support. In spite of this limitation, it is worth examining the *RSR* system in some detail since it demonstrates the logic involved in developing a quasi-quantitative rock mass classification system.

The significance of the *RSR* system, in the context of this discussion, is that it introduced the concept of rating each of the components listed below, i.e., A, B & C to arrive at a numerical value of *RSR* by finding the sum of A, B & C.

$$RSR = A + B + C$$

1. *Parameter A —Geology:* General appraisal of geological structure on the basis of:
   a) Rock type origin (igneous, metamorphic, sedimentary)
   b) Rock hardness (hard, medium, soft, decomposed)
   c) Geologic structure (massive, slightly faulted/folded, moderately faulted/folded, intensely faulted/folded)
2. *Parameter B—Geometry*: Effect of discontinuity pattern with respect to the direction of the tunnel drive on the basis of:
   a) Joint spacing
   b) Joint orientation (strike and dip)
   c) Direction of tunnel drive
3. *Parameter C* —Effect of groundwater inflow and joint condition on the basis of:
   a) Overall rock mass quality on the basis of A and B combined.
   b) Joint condition (good, fair, poor).
   c) Amount of water inflow (in gallons per minute per 1000 feet of tunnel).

Note that the *RSR* classification used Imperial units and that these units have been retained in this discussion.

Three tables from the paper of Wickham et al. (1972) are reproduced in Tables 4.1, 4.2 and 4.3. These tables can be used to evaluate the rating of each of these parameters to arrive at the *RSR* value (maximum *RSR* = 100).

Table 4.1: Rock Structure Rating

Average Joint Spacing	Strike ⊥ to Axis					Strike || to Axis		
	Direction of Drive					Direction of Drive		
	Both	With Dip		Against Dip		Either Direction		
	Dip of Prominent Joints[a]					Dip of Prominent Joints		
	Flat	Dipping	Vertical	Dipping	Vertical	Flat	Dipping	Vertical
1. Very closely jointed, < 2 in	9	11	13	10	12	9	9	7
2. Closely jointed, 2–6 in	13	16	19	15	17	14	14	11
3. Moderately jointed, 6–12 in	23	24	28	19	22	23	23	19
4. Moderate to blocky, 1–2 ft	30	32	36	25	28	30	28	24
5. Blocky to massive, 2–4 ft	36	38	40	33	35	36	24	28
6. Massive, > 4 ft	40	43	45	37	40	40	38	34

Table 4.2: Rock Structure Rating of Parameter *B* —Joint Pattern, Direction of Drive

	Basic Rock Type				Geological Structure			
	Hard	Medium	soft	Decomposed				
Igneous	1	2	3	4	Massive	Slightly Folded or Faulted	Moderately Folded or Faulted	Intensively Folded or Faulted
Metamorphic	1	2	3	4				
Sedimentary	2	3	4	4				
Type 1					30	22	15	9
Type 2					27	20	13	8
Type 3					24	18	12	7
Type 4					19	15	10	6

Table 4.3: Rock Structure Rating of Parameter *C* — Groundwater, Joint Condition

Anticipated Water Inflow gpm/1000 ft of tunnel	Sum of Parameters A +B					
	13–44			45–75		
	Joint Condition [b]					
	Good	Fair	Poor	Good	Fair	Poor
None	22	18	12	25	22	18
Slight (< 200 gpm)	19	15	9	23	19	14
Moderate (200–1000 gpm)	15	22	7	21	16	12
Heavy (>1000 gpm)	10	8	6	18	14	10

a) Dip — flat: 0^0-20^0; dipping: 20^0-50^0; and vertical: 50^0-90^0

b) Joint condition — good = tight or cemented; fair = slightly weathered or altered; poor = severely weathered, altered or open joint.

For example, a hard metamorphic rock which is slightly folded or faulted has a rating of A = 22 (from Table 4.2). The rock mass is moderately jointed, with joints striking perpendicular to the tunnel axis which is being driven east-west, and dipping at between 20° and 50°.

A typical set of prediction curves for a 24-foot diameter tunnel is given in Figure 4.2 which shows that, for the *RSR* value of 62 derived above, the predicted support would be 2 inches of shotcrete and 1 inch diameter rockbolts spaced at 5 foot centres. As indicated in the Figure, steel sets would be spaced at more than 7 feet apart and would not be considered a practical solution for the support of this tunnel.

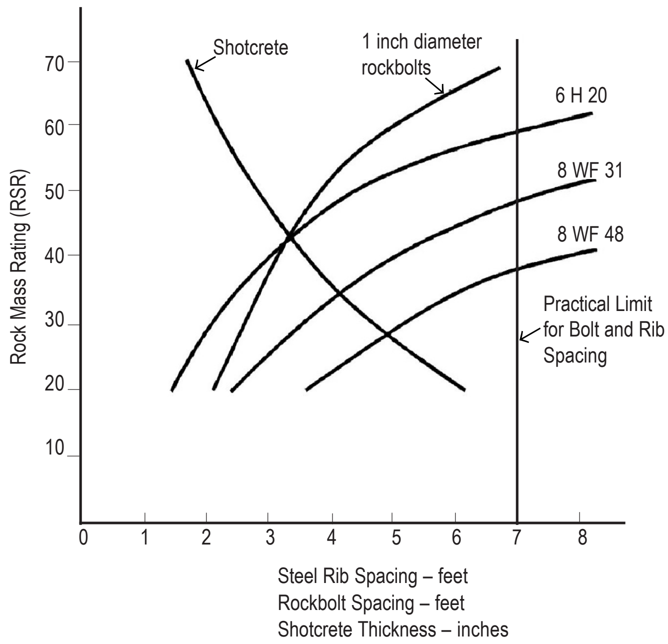


Figure 4.2: RSR *Support Estimates for a 24 ft (7.3 m) Diameter Circular Tunnel. Note that Rockbolts and Shotcrete are Generally Used Together. (After Wickham et al. 1972)*

For the same size tunnel in a rock mass with *RSR* = 30, the support could be provided by 8 WF 31 steel sets (8 inch deep wide flange I section weighing 31 lb per foot) spaced 3 feet apart, or by 5 inches of shotcrete and 1 inch diameter rockbolts spaced at 2.5 feet centres. In this case, it is probable that the steel set solution would be cheaper and more effective than the use of rockbolts and shotcrete.

Although the *RSR* classification system is not widely used today, the work of Wickham et al. played a significant role in the development of the classification schemes discussed in the remaining sections of this Chapter.

4.3 GEOMECHANICS CLASSIFICATION

Bieniawski (1976) published the details of a rock mass classification called the Geomechanics Classification or the Rock Mass Rating (*RMR*) system. Over the years, this system has been successively refined as more case records have been examined and the reader should be aware that Bieniawski has made significant changes in the ratings assigned to different parameters. The discussion which follows is based upon the 1989 version of the classification (Bieniawski, 1989). Both, this version and the 1976 version, deal with estimating the strength of rock masses. The following six parameters are used to classify a rock mass using the *RMR* system:

a) Uniaxial compressive strength of rock material

b) Rock Quality Designation (*RQD*)

c) Spacing of discontinuities

d) Condition of discontinuities

e) Groundwater conditions

f) Orientation of discontinuities

In applying this classification system, the rock mass is divided into a number of structural regions and each region is classified separately. The boundaries of the structural regions usually coincide with a major structural feature such as a fault or with a change in rock type. In some cases, significant changes in discontinuity spacing or characteristics, within the same rock type, may necessitate the division of the rock mass into a number of small structural regions.

The Rock Mass Rating system is presented in Table 4.4, giving the ratings for each of the six parameters listed above. These ratings are summed to give a value of *RMR*. The following example illustrates the use of these tables to arrive at an *RMR* value.

Bieniawski (1989) published a set of guidelines for the selection of support in tunnels in rock for which the value of *RMR* has been determined. These guidelines are reproduced in Table 4.5. Note that these guidelines have been published for a 10 m span horseshoe-shaped tunnel, constructed using drill and blast methods, in a rock mass subjected to a vertical stress < 25 MPa (equivalent to a depth below surface of <900 m). For the case considered earlier, with *RMR* = 59, Table 4.5 suggests that a tunnel could be excavated by top heading and bench, with a 1.5 to 3 m advance in the top heading.

Support should be installed after each blast and the support should be placed at a maximum distance of 10 m from the face. Systematic rock bolting, using 4 m long & 20 mm diameter fully grouted bolts spaced at 1.5 to 2 m in the crown and walls, is recommended. Wire mesh, with 50 to 100 mm of shotcrete for the crown and 30 mm of shotcrete for the walls, is recommended.

The value of *RMR* of 59 indicates that the rock mass is on the boundary between the 'Fair rock' and 'Good rock' categories. In the initial stages of design and construction, it is advisable to utilise the support suggested for fair rock. If the construction is progressing well with no stability problems, and the support is performing very well, then it should be possible to gradually reduce the support requirements to those indicated for a good rock mass. In addition, if the excavation is required to be stable for a short amount of time, then it is advisable to try the less expensive and extensive support suggested for good rock. However, if the rock mass surrounding the excavation is expected to undergo large mining induced stress changes, then more substantial support appropriate for fair rock should be installed. This example indicates that a great deal of judgement is needed in the application of rock mass classification to support design.

The *RMR* value for the example under consideration is determined as follows:

Table	Item	Value	Rating
4: A.1	Point Load Index	8 MPa	12
4: A.2	RQD	70%	13
4: A.3	Spacing of Discontinuities	300 mm	10
4: E. 4	Conditions of Discontinuities	Note 1	22
4: A.5	Groundwater	Wet	7
4: B	Adjustment for Joint Orientation	Note 2	- 5
		Total	**59**

Table 4.4: Rock Mass Rating System (After Bieniawski 1989) : ISRM

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

	Parameters		Ranges of Values						
1.	Strength of intact rock material	Point-load strength index (MPa)	>10	4–10	2–4	1–2	For this low range, uniaxial compressive test is preferred		
		Uniaxial compressive strength (MPa)	>250	100–250	50–100	25–50	5–25	1–5	<1
	Rating		>15	12	7	4	2	1	0
2.	Drill quality RQD (%)		90–100	75–90	50–75	25–50	25		
	Rating		20	17	13	8	3		
3.	Spacing of discontinuities		>0.2 m	0.06–0.2 m	200–600 mm	200–600 mm	<60 mm		
	Rating		20	15	10	8	5		
4.	Condition of discontinuities		Very rough surfaces; Not continuous. No separation; Unweathered wall rock.	Slightly rough surfaces; Separation <1 mm; Slightly weathered wall.	Slightly rough surfaces; Separation <1 mm; Highly weathered wall.	Slickensided Surface or Gouge <5 mm thick or Separation 1–5 mm contours	Soft gouge > 5 mm thick or Separation >5 mm contours		
	Rating		30	25	20	10	0		
5.	Groundwater	Inflow per 10 m tunnel length (min.)	None	<10	10–25	25–125	>125		
		(Joint water Press)/ (Measure principal σ)	0	<0.1	0.1–0.2	0.2–0.5	>0.5		
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing		
	Rating		15	10	7	4	0		

B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS

Strike and Dip Orientations of Discontinuities		Very Favourable	Favourable	Fair	Unfavourable	Very Unfavourable
Rating	Tunnels and Mines	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	-60

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

Rating	100–81	80–61	60-41	40-21	<20
Class no.	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

D. MEANING OF ROCK CLASSES

Class no.	I	II	III	IV	V
Average stand-up time	20yr for 15m span	1 yr for 10 m span	1wk for 5m span	10 h for 2.5m span	30 min for 1m span
Cohesion of the rock mass (kPa)	>400	300–400	200–300	100–200	<100
Friction angle of the rock mass(deg)	>45	35–45	25–35	15–25	<15

E. GUIDELINES FOR CLASSIFICATION OF DISCONTINUITY CONDITIONS

Discontinuity length (persistence)	<1 m	1–3 m	3–10 m	10–20 m	>20 m
Rating	6	4	2	5	0
Separation (aperture)	None	<0.1 mm	0.1–1.0 mm	1–5 mm	>5 mm
Rating	6	5	4	1	0
Roughness	Very rough	Rough	Slightly rough	Smooth	Slickensided
Rating	6	5	3	1	0
Infilling (gauge)	None	Hand filling <5 mm	Hand filling >5 mm	Soft filling <5 mm	Soft filling >5 mm
Rating	6	4	2	2	0
Weathering	Unweathered	Slightly weathered	Moderately weathered	Highly weathered	Decomposed
Rating	6	5	3	1	0

F. EFFECT OF DISCONTINUITY STRIKE AND DIP OREINTATION TUNNELLING**

Strike perpendicular to tunnel axis		Strike parallel to tunnel axis	
Drive with dip — Dip 45–90^0	Drive with dip — Dip 20–45^0	Dip 45–90^0	Dip 20–45^0
Very favourable	Favourable	Very unfavourable	Fair
Drive against dip — Dip 45–90^0	Drive against dip — Dip 20–45^0	Dip 0–20^0 irrespective of strike	
Fair	Unfavourable	Fair	

* Some conditions are mutually exclusive. For example, if infilling is present, the roughness of the surface will be overshadowed by the influence of the gouge. In such cases use A.4 directly; ** Modified after Wickham et al. (1972)

Table 4.5: Guidelines for Excavation and Support of 10 m Span Rock Tunnels in accordance with the *RMR* system (After Bieniawski 1989)

Rock Mass Class	Excavation	Rock Bolts (20 mm diameter, fully grouted)	Shotcrete	Steel Sets
I - Very good rock *RMR:* 81–100	Full face, 3 m advance	Generally no support required except spot bolting		
II - Good rock *RMR:* 61–80	Full face, 1–1.5 m advance. Complete support 20 m from face	Locally, bolts in crown 3 m long, spaced 2.5 m with occasional wire mesh	50 mm in crown where required	None
III - Fair rock *RMR:* 41–60	Top heading and bench 1.5–3 m advance in top heading. Commence support after each blast. Complete support 10 m from face.	Systematic bolts 4 m long, spaced 1.5–2 m in crown and walls with wire mesh in crown	50–100 mm in crown and 30 mm in sides	None
IV - Poor rock *RMR:* 21–40	Top heading and bench 1.0–1.5 m advance in top heading. Install support concurrently with excavation, 10 m from face.	Systematic bolts 4–5 m long, spaced 1–1.5 m in crown and walls with wire mesh.	100–150 mm in crown and 100 mm in sides	Light to medium ribs spaced 1.5 m where required
V - Very poor rock *RMR:* <20	Multiple drifts 0.5–1.5 m advance in top heading. Install support concurrently with excavation. Shotcrete as soon as possible after blasting.	Systematic bolts 5–6 m long, spaced 1–1.5 m in crown and walls with wire mesh. Bolt invert.	100–200 mm in crown and 150 mm in sides and 50 mm on face.	Medium to heavy ribs spaced 0.75 m with steel lagging and forepoling if required. Close invert.

It should be noted that Table 4.5 has not had a major revision since 1973. In many mining and civil engineering applications, steel fibre reinforced shotcrete may be considered in place of wire mesh and shotcrete.

4.4 MODIFICATIONS TO *RMR* FOR MINING APPLICATIONS

Bieniawski's Rock Mass Rating (*RMR*) system was originally based upon case histories drawn from civil engineering. Consequently, the Mining Industry tended to regard the classification as tad conservative and several modifications were proposed in order to make the classification more relevant to mining applications. A comprehensive summary of these modifications was compiled by Bieniawski (1989).

Laubscher (1977, 1984), Laubscher and Taylor (1976) and Laubscher and Page (1990) have described a Modified Rock Mass Rating system for mining. This MRMR system takes the basic *RMR* value, as defined by Bieniawski, and adjusts it to account for in situ and induced stresses, stress changes and the effects of blasting and weathering. A set of support recommendations is associated with the resulting *MRMR* value. In using Laubscher's *MRMR* system it should, however, be borne in mind that many of the case histories upon which it is based are derived from caving operations. Originally, block caving in asbestos mines in Africa formed the basis for the modifications but, subsequently, other case histories from around the world were added to the database.

Cummings et al (1982) and Kendorski et al. (1983) have also modified Bieniawski's RMR classification to produce the *MBR* (modified basic *RMR*) system for mining. This system was developed for block caving operations in the USA. It involves the use of different ratings for the original parameters used to determine the value of *RMR* and the subsequent adjustment of the resulting *MBR* value to allow for blast damage, induced stresses, structural features, distance from the cave front and size of the caving block.

Support recommendations are presented for isolated or development drifts as well as for the final support of intersections and drifts.

4.5 ROCK TUNNELLING QUALITY INDEX

On the basis of an evaluation of a large number of case histories of underground excavations, Barton et al. (1974) of the Norwegian Geotechnical Institute proposed a Tunnelling Quality Index (Q) for the determination of rock mass characteristics and tunnel support requirements. The numerical value of the index Q varies on a logarithmic scale from 0.001 to a maximum of 1,000 and is defined by:

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \qquad (2)$$

where, RQD is the Rock Quality Designation

J_n is the joint set number

J_r is the joint roughness number

J_a is the joint alteration number

J_w is the joint water reduction factor

SRF is the stress reduction factor

In explaining the meaning of the parameters used to determine the value of Q, Barton et al. (1974) offered the following comments:

The first quotient (RQD/J_n), representing the structure of the rock mass, is a crude measure of the block or particle size, with the two extreme values (100/0.5 and 10/20) differing by a factor of 400. If the quotient is interpreted in units of centimetres, the extreme 'particle sizes' of 200 to 0.5 cm are seen to be crude but fairly realistic approximations. Probably the largest blocks should be several times this size and the smallest fragments less than half the size. (Clay particles are of course excluded).

The second quotient (J_r/J_a) represents the roughness and frictional characteristics of the joint walls or filling materials. This quotient is weighted in favour of rough, unaltered joints in direct contact. It is to be expected that such surfaces will be close to peak strength, that they will dilate strongly when sheared, and they will therefore be especially favourable to tunnel stability.

When rock joints have thin clay mineral coatings and fillings, the strength is reduced significantly. Nevertheless, rock wall contact after small shear displacements have occurred, may be a very important factor for preserving the excavation from ultimate failure.

Where no rock wall contact exists, the conditions are extremely unfavourable to tunnel stability. The 'friction angles' (given in Table 4.6) are a little below the residual strength values for most clays, and are possibly down-graded by the fact that these clay bands or fillings may tend to consolidate during shear, at least if normal consolidation or if softening and swelling has occurred. The swelling pressure of montmorillonite may also be a factor here.

The third quotient (*Jw/SRF*) consists of two stress parameters. *SRF* is a measure of —a) loosening load in the case of an excavation through shear zones and clay bearing rock, b) rock stress in competent rock and c) squeezing loads in plastic incompetent rocks. It can be regarded as a total stress parameter. The parameter J_w is a measure of water pressure, which has an adverse effect on the shear strength of joints due to a reduction in effective normal stress. Water may, in addition, cause softening and possible outwash in the case of clay-filled joints. It has proved impossible to combine these two parameters in terms of inter-block effective stress, because paradoxically a high value of effective normal stress may sometimes signify less stable conditions than a low value, despite the higher shear strength. The quotient (J_w/SRF) is a complicated empirical factor describing the 'active stress'.

It appears that the rock tunnelling quality *Q* can now be considered to be a function of only three parameters which are crude measures of:

(i) Block size (RQD/J_n)

(ii) Inter-block shear strength (J_r/J_a)

(iii) Active stress (J_w/SRF)

Undoubtedly, there are several other parameters which could be added to improve the accuracy of the classification system. One of these would be the joint orientation. Although many case records include the necessary information on structural orientation in relation to excavation axis, it was not found to be the important general parameter that might be expected. Part of the reason for this may be that the orientations of many types of excavations can be, and normally are, adjusted to avoid the maximum effect of unfavourably oriented major joints. However, this choice is not available in the case of tunnels, and more than half the case records were in this category. The parameters J_n, J_r and J_a appear to play a more important role than orientation, because the number of joint sets determines the degree of freedom for block movement (if any), and the frictional and dilational characteristics can vary more than the down-dip gravitational component of unfavourably oriented joints. If joint orientations had been included the classification would have been less general, and its essential simplicity lost.

Table 4.6 (After Barton et al. 1974) gives the classification of individual parameters used to obtain the Tunnelling Quality Index *Q* for a rock mass. The use of Table 4.6 is illustrated in the following example. A 15 m span crusher chamber for an underground mine is to be excavated in a norite at a depth of 2,100 m below surface. The rock mass contains two sets of joints controlling stability. These joints are undulating, rough and unweathered with very minor surface staining. *RQD* values range from 85% to 95% and laboratory tests on core samples of intact rock give an average uniaxial compressive strength of 170 MPa. The principal stress directions are approximately vertical and horizontal and the magnitude of the horizontal principal stress is approximately 1.5 times that of the vertical principal stress. The rock mass is locally damp but there is no evidence of flowing water.

The numerical value of *RQD* is used directly in the calculation of *Q* and, for this rock mass, an average value of 90 will be used. Table 4.6 shows that, for two joint sets, the joint set number, $J_n = 4$. For rough or irregular joints which are undulating, Table 4.6 gives a joint roughness number of $J_r = 3$. Table 4.6 gives the joint alteration number, $J_a = 1.0$, for unaltered joint walls with surface staining only. Table 4.6 shows that, for an excavation with minor inflow, the joint water reduction factor, $J_w = 1.0$. For a depth below the surface of 2,100 m the overburden stress will be approximately 57 MPa and, in this case, the major principal stress $\sigma_1 = 85$ MPa. Since the uniaxial compressive strength of the norite is approximately 170 MPa, this gives a ratio of $\sigma_c/\sigma_1 = 2$. Table 4.6 shows that, for competent rock with rock stress problems, this value of σ_c/σ_1 can be expected to produce heavy rock burst conditions and that the value of *SRF* should lie between 10 and 20. A value of *SRF* = 15 will be assumed for this calculation. Using these values gives:

Table 4.6 : Classification of Individual Parameters Used in the Tunnelling Quality Index *Q*

DESCRIPTION	VALUE	NOTES	
1. ROCK QUALITY DESIGNATION	RQD		
A. Very poor	0–25	1. Where RQD is reported or measured as ≤ 10 (including 0), a nominal value of 10 is used to evaluable Q.	
B. Poor	25–50		
C. Fair	50–75		
D. Good	75–90	2. RQD intervals of 5, i.e., 100, 95, 90 etc. are sufficiently accurate.	
E. Excellent	90–100		
2. JOINT SET NUMBER	J_n		
A. Massive, no or few joints	0.5–1.0		
B. One joint set	2		
C. One joint set plus random	3		
D. Two joint sets	4		
E. Two joint sets plus random	6		
F. Three joint sets	9	1. For intersections use (3.0 x J_n)	
G. Three joint sets plus random	12		
H. Four or more joint sets, random, heavily jointed, 'sugar cube', etc.	15	2. For portals use (2.0 X J_n)	
I. Crushed rock, earthlike	20		
3. JOINT ROUGHNESS NUMBER	J_r		
a) Rock wall contact			
b) Rock wall contact before 10 cm shear			
A. Discontinuous joints	4		
B. Rough and irregular, undulating	3		
C. Smooth undulating	2		
D. Slickensided undulating	1.5	1. Add 1.0 if the mean spacing of the relevant joint set is greater than 3 m.	
E. Rough or irregular, planar	1.5		
F. Smooth Planar	1.0		
G. Slickensided, planar	0.5	2. J_r = 0.5 can be used for planar, slickensided joints having lineations, provided that the lineations are oriented for minimum strength.	
c) No rock wall contact when sheared			
H. Zones containing clay minerals thick enough to prevent rock wall contact	1.0 (nominal)		
I. Sandy, gravely or crushed zone thick enough to prevent rock wall contact	1.0 (nominal)		
4. JOINT ALTERATION NUMBER	J_a	*Ör* degrees (approx.)	
a) Rock wall contact			
A. Tightly healed, hard, non-softening, Impermeable filling	0.75		1. Values of *Ör*, the residual friction angle, are intended as an approximate guide to the mineralogical properties of the alteration products, if present.
B. Unaltered joint walls, surface staining only	1.0	25– 35	
C. Slightly altered joint walls, non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc.	2.0	25–30	
D. Silty, or sandy-clay coatings, small clay fraction (non-softening)	3.0	20–25	
E. Softening or low-friction clay mineral coatings, i.e. kaolinite, mica. Also chlorite, talc, gypsum and graphite etc., and small quantities of swelling clays. (Discontinuous coatings, 1–2 mm or less)	4.0	8–16	

contd...

DESCRIPTION	VALUE	NOTES
b) Rock wall contact before 10 cm shear	J_a	*Ör* degrees (approx.)
F. Sandy particles, clay-free, disintegrating rock etc.	4.0	25–30
G. Strongly over-consolidated, non-softening clay mineral fillings (continuous < 5mm thick)	6.0	16–24
H. Medium or low over-consolidation, softening clay mineral filling (continuous < 5mm thick)	8.0	12–16
I. Swelling clay fillings, i.e., monthmorillonite, (continuous < 5mm thick). Values of J_a depend on percent of swelling clay-size particles and access to water	8.0–12.0	6–12
c. No rock wall contact when sheared		
J. Zones or bands of disintegrated or crushed	6.0	
K. rock and clay (see G,H and J for clay)	8.0	
L. conditions)	8.0–12.0	6–24
M. Zones or bands of silty or sandy-clay, small clay fraction, non—softening	5.0	
N. Thick continuous zones or bands of clay	10.0–13.0	
O. & R (see G.H. and J for clay conditions)	6.0–24.0	
5. JOINT WATER REDUCTION	*Jw*	Apporx water pressure (kgf/cm^2)
A. Dry excavation or minor inflow i.e. <5 l/m locally	1.0	<1.0
B. Medium inflow or pressure, occasional outwash of joint fillings	0.66	1.0–2.5
C. Large inflow or high pressure in competent rock with unfilled joints	0.5	2.5–10.0 1. Factors C to F are crude estimates; increases J_w if drainage installed.
D. Large inflow or high pressure	0.33	2.5–10.0
E. Exceptionally high inflow or pressure at blasting , decaying with time	0.2–0.1	>10 2. Special problems caused by ice formation are not considered.
F. Exceptionally high inflow or pressure	0.1–0.05	>10
6. STRESS REDUCING FACTOR		**SRF**
a) Weakness zones intersecting excavation , which may cause loosening of rock mass when tunnel is excavated		
A. Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock any depth)		10.0 1. Reduce these values of SRF by 25-50% but only if the relevant shear zones influence do not intersect the excavation
B. Single weakness zones containing clay, or chemically disintegrated rock (excavated depth <50 m)		5.0
C. Single weakness zones containing clay, or chemically disintegrated rock (excavation depth > 50 m)		2.5

contd...

DESCRIPTION	VALUE			NOTES
D. Multiple shear zones in competent rock (clay free), loose surrounding rock (any depth)			7.5	
E. Single shear zone in competent rock (clay free) (depth of excavation > 50 m)			5.0	
F. Single shear zone in competent rock (clay free). (depth of excavation > 50m)			2.5	
G. Loose open joints, heavily jointed or 'sugar cube', (any depth)			5.0	
b) Competent rock, rock stress problems	σ_c/σ_1	σ_t/σ_1		
H. Low stress, rear surface	>200	>13	2.5	2. For strongly anisotropic virgin stress field (if measured) : when $5 \leq \sigma_1/\sigma_3 \leq 10$, reduce σ_c to 0.8 σ_c and σ_t to 0.8 σ_t. When $\sigma_1/\sigma_3 >$ 10, reduce σ_c and σ_t to 0.6 σ_c and 0.6 σ_t, where σ_c = unconfined compressive strength and σ_t = tensile strength (point load) and σ_1 and σ_3 are the major and minor principal stresses.
I. Medium stress	200–10	13–0.66	1.0	
J. High stress, very tight structure (usually favorable to stability, may be unfavorable to wall stability)	10-5	0.66–0.33	0.5–2	
K. Mild rockburst (massive rock)	5–2.5	0.33–0.16	5–10	
L. Heavy rockburst (massive rock)	<2.5	<0.16	10–20	3. Few case records available where depth of crown below surface is less than span width. Suggest SRF increases from 2.5 to 5 for such cases (see H).
c) Squeezing rock, plastic flow of incompetent rock under influence of high rock pressure				
M. Mild squeezing rock pressure			5–10	
N. Heavy squeezing rock pressure			10–20	
d) Swelling rock , chemical swelling activity depending on presence of water				
O. Mild swelling rock pressure			5–10	
P. Heavy swelling rock pressure			10–15	

ADDITIONAL NOTES ON THE USE OF THESE TABLES

When making estimates of the rock mass Quality (Q), the following guidelines should be followed in addition to the notes listed in the tables:

1. When borehole core is unavailable, RQD can be estimated from the number of joints per unit volume, in which the number of joints per metre for each joint set are added. A simple relationship can be used to convert this number to RQD for the case of clay free rock masses: RQD = 115- 3.3 J_v (approx), where J_v = total number of joints per m^3 (o < RQD < 100 for 35> J_v > 4.5).
2. The parameter J_n representing the number of joints sets will often be affected by foliation, schistosity, slaty cleavage or bedding etc. If strongly developed, these parallel 'joints' should obviously be counted as a complete joint set. However, if there are few 'joints' visible , or if only occasional breaks in the core are due to these features , then it will be more appropriate to count them as 'random' joints when evaluating J_n.
3. The parameter J_r and J_a (representing shear strength) should be relevant to the weakest significant joint set or clay filled discontinuity in the given zone. However, if the joint or discontinuity with the minimum value of J_rIJ_a is favorably oriented for stability, then a second, less favorably oriented joint set or discontinuity may sometimes be more significant and its higher value of J_rIJ_a should be used when evaluating Q. The value of J_rIJ_a should in fact relate to the surface most likely to allow failure to initiate.
4. When a rock mass contains clay, the factor SRF appropriate to loosening loads should be evaluated. In such cases the strength of the intact rock is of little interest. However when joining is minimal and clay is completely absent, the strength of the intact rock may become the weakest link and the the stability will then depend on the ratio rock –stress/rock-strength. A strongly anisotropic stress field is unfavorable for stability and is roughly accounted for as in note 2 in the table for stress reduction factor evaluation.
5. The compressive and tensile strengths (ó$_c$ and ó$_t$) of the intact rock should be evaluated in the saturated condition if this is appropriate to the present and future in situ conditions. A very conservative estimate of the strength should be made for those rocks that deteriorate when exposed to moist or saturated conditions.

$$Q = \frac{90}{4} \times \frac{3}{1} \times \frac{1}{15} = 4.5 \qquad (3)$$

In relating the value of the index Q to the stability and support requirements of underground excavations, Barton et al. (1974) defined an additional parameter which they called the Equivalent Dimension, D_e, of the excavation. This dimension is obtained by dividing the span, diameter or wall height of the excavation by a quantity called the Excavation Support Ratio, *ESR*. Hence:

D_e = Excavation span, diameter or height (m) / Excavation Support Ratio *ESR*

The value of *ESR* is related to the intended use of the excavation and to the degree of security which is demanded of the support system installed to maintain the stability of the excavation. Barton et al. (1974) suggest the following values:

Excavation Category		ESR
A	Temporary mine openings	3-5
B	Permanent mine openings, water tunnels for hydro power (excluding high pressure penstocks), pilot tunnels, drifts and headings for large excavations	1.6
C	Storage rooms, water treatment plants, minor road and railway tunnels, surge chambers, access tunnels.	1.3
D	Power stations, major road and railway tunnels, civil defence chambers, portal intersections.	1.0
E	Underground nuclear power stations, railway stations, sports and public facilities, factories	0.8

Barton et al. (1980) provide additional information on rockbolt length, maximum unsupported spans and roof support pressures to supplement the support recommendations published in the original 1974 paper. The Estimated support categories based Tunnelling Quality Index Q are shown in Figure 4.3.

The length L of rockbolts can be estimated from the excavation width B and the Excavation Support Ratio *ESR*:

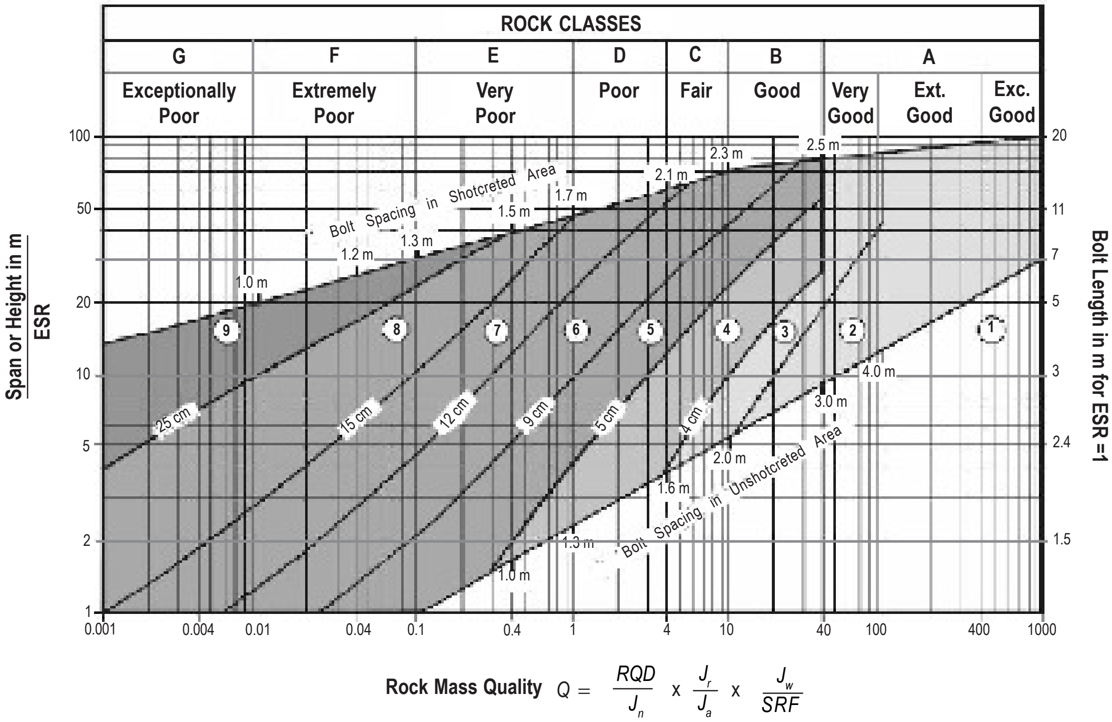
$$L = 2 + \frac{0.15B}{ESR} \qquad (4)$$

The maximum unsupported span can be estimated from:

Maximum span (unsupported) = $2ESR\ Q^{0.4}$ (5)

Based upon analyses of case records, Grimstad and Barton (1993) suggest that the relationship between the value of Q and the permanent roof support pressure *Proof* is estimated from:

$$P_{roof} = \frac{2\sqrt{J_n}\, Q^{-\frac{1}{3}}}{3J_r} \qquad (6)$$



REINFORCEMENT CATEGORY

1. Unsupported
2. Spot Bolting
3. Systematic Bolting
4. Systematic Bolting (and unreinforced shotcrete, 4–10 cm)
5. Fibre Reinforced Shotcrete and Bolting, 5–9 cm
6. Fibre Reinforced Shotcrete and Bolting, 9–12 cm
7. Fibre Reinforced Shotcrete and Bolting, 12–15 cm
8. Fibre Reinforced Shotcrete, > 15 cm, Reinforced Ribs of Shotcrete and Bolting
9. Cost Concrete Lining

Figure 4.3: Estimated Support Categories Based on the Tunnelling Quality Index *Q* (After Grimstad and Barton, 1993, Reproduced from Palmstrom and Broch, 2006)

4.6 USING ROCK MASS CLASSIFICATION SYSTEMS

The two most widely used rock mass classifications are Bieniawski's *RMR* (1976, 1989) and Q-system of Barton et al. (1974). Both methods incorporate geological, geometric and design/engineering parameters in arriving at a quantitative value of their rock mass quality. The similarities between *RMR* and *Q*-system are the use of identical or very similar parameters in calculating the final rock mass quality rating. The differences between the systems lie in the different weightings given to similar parameters and in the use of distinct parameters in one or the other scheme.

RMR uses compressive strength directly while *Q* only considers strength as it relates to in situ stress in competent rock. Both schemes deal with the geology and geometry of the rock mass, but in slightly different ways. Both consider groundwater, and both include some component of rock material strength. Some estimate of orientation can be incorporated into *Q* using a guideline presented by Barton et al. (1974)— 'the parameters J_r and J_a relate to the surface most likely to allow failure to initiate.' The greatest difference between the two systems is the lack of a stress parameter in the *RMR* system.

When using either of these methods, two approaches can be taken. One is to evaluate the rock mass specifically for the parameters included in the classification methods; the other is to accurately characterise the rock mass and then attribute parameter ratings at a later time. The latter method is recommended since it gives a full and complete description of the rock mass which can easily be translated into either classification index. If rating values alone had been recorded during mapping, it would be almost impossible to carry out verification studies.

In many cases, it is appropriate to give a range of values to each parameter in a rock mass classification and to evaluate the significance of the final result. The average value of Q can be used in choosing a basic support system while the range gives an indication of the possible adjustments which will be required to meet different conditions encountered during construction.

A further example of this approach is given in a paper by Barton et al. (1992) concerned with the design of a 62 m span underground sports hall in jointed gneiss. Histograms of all the input parameters for the Q-system are presented and analysed in order to determine the weighted average value of Q. Carter (1992) has adopted a similar approach, but extended his analysis to include the derivation of a probability distribution function and the calculation of a probability of failure in a discussion on the stability of surface crown pillars in abandoned metal mines.

Throughout this chapter it has been suggested that the user of a rock mass classification scheme should check that the latest version is being used. It is also worth repeating that the use of two rock mass classification schemes side by side is advisable.

Geo-mechanics classification will provide the guidelines for selection of supports to ensure the long stability of various rock classes. For Maximum RMR Value, Unsupported Span may be worked out using Figure 4.4.

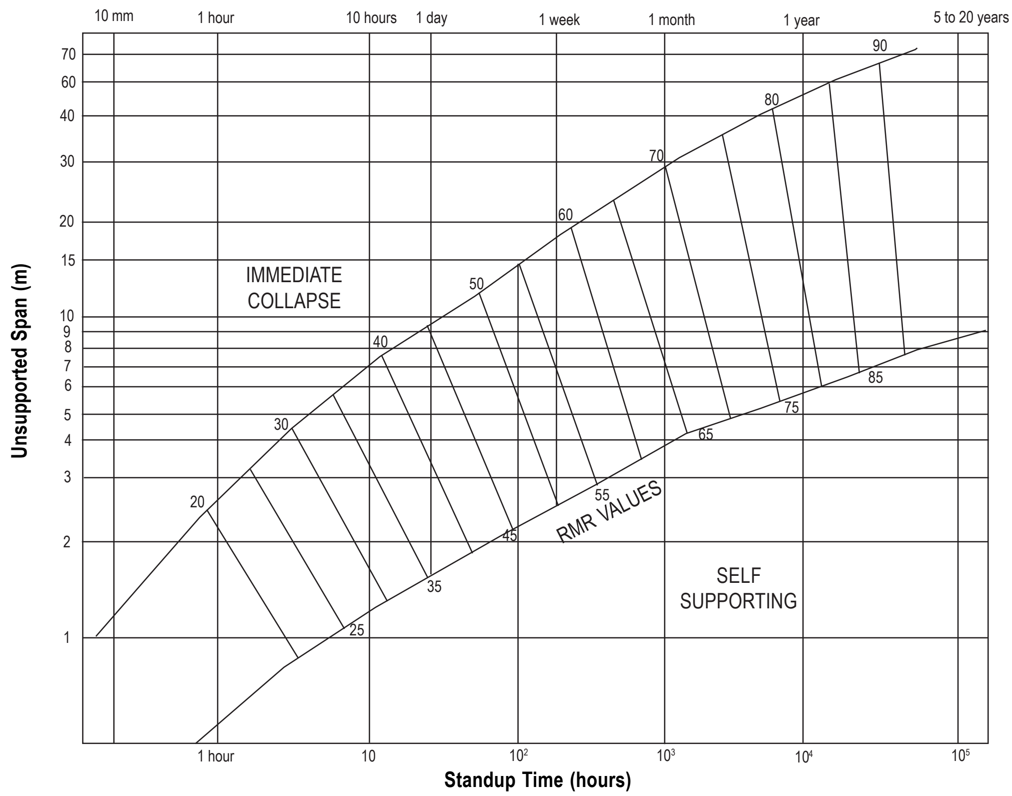


Figure 4.4: Relation Between Unsupported Span, Standup Time and RMR Values

4.7 MINING ROCK MASS RATING CLASSIFICATION (MRMR)

Laubscher developed the Mining Rock Mass Rating (MRMR) system by modifying the Rock Mass Rating (RMR) system of Bieniawski. In the MRMR system, the stability and support are determined with the following equations:

RMR = IRS + RQD + spacing + condition

in which:

RMR	=	Laubschers Rock Mass Rating
IRS	=	Intact Rock Strength
RQD	=	Rock Quality Designation
spacing	=	expression for the spacing of discontinuities
condition	=	condition of discontinuities (parameter also dependent on ground water presence, pressure, or quantity of groundwater inflow in the underground excavation)

MRMR = RMR * adjustment factors

in which:

adjustment factors	=	factors to compensate for — the method of excavation, orientation of discontinuities and excavation, induced stresses, and future weathering

The parameters to calculate the RMR value are similar to those used in the RMR system of Bieniawski. This may be confusing, as some of the parameters in the MRMR system are modified, such as the condition parameter that includes groundwater presence and pressure in the MRMR system whereas groundwater is a separate parameter in the RMR system of Bieniawski. The number of classes for the parameters and the detail of the description of the parameters are also more extensive than in the RMR system of Bieniawski.

The adjustment factors depend on future (susceptibility to) weathering, stress environment, orientation, etc.

The combination of values of RMR and MRMR determines the so-called reinforcement potential. A rock mass with a high RMR before the adjustment factors are applied has a high reinforcement potential, and can be reinforced by, for example, rock bolts, whatever the MRMR value might be after excavation. Contrariwise, rock bolts are not a suitable reinforcement for a rock mass with a low RMR (i.e., has a low reinforcement potential).

Laubscher uses a graph for the spacing parameter. The parameter is dependent on a maximum of three discontinuity sets that determine the size and the form of the rock blocks. The condition parameter is determined by the discontinuity set with the most adverse influence on the stability.

The concept of adjustment factors for the rock mass before and after excavation is very attractive. This allows for compensation of local variations, which may be present at the location of the rock mass observed, but might not be present at the location of the proposed excavation or vice versa. In addition, this allows for quantification of the influence of excavation and excavation induced stresses, excavation methods, and the influence of past and future weathering of the rock mass.

● ● ●

Chapter

5

Slope Stability

5.0 DESIGN OF EXCAVATIONS — OPENCAST PIT SLOPE DESIGN GUIDELINES

Slope stability has significant importance in open-pit excavation for the purpose of profitability. This factor affects— a) formation, contacts, structural factors of deposit; b) rock type, intrusive; c) joints and discontinuities; d) existing slope design; and e) condition of the existing benches due to erosion.

To achieve optimum slope design, visual observation of the condition of benches followed by engineering judgments, rock testing, geomechanic classification, stereo net analysis and numerical analysis need to carry out.

Aim of Slope Stability

- To understand the development and form of natural and man-made slopes and the processes responsible for different features.
- To assess the stability of slopes under short-term (often during construction) and long-term conditions.
- To assess the possibility of slope failure involving natural or existing engineered slopes.
- To analyse slope stability and to understand failure mechanisms and the influence of environmental factors.
- To enable the redesign of failed slopes and the planning and design of preventive and remedial measures, where necessary.
- To study the effect of seismic loadings on slopes and embankments.
- To evolve safe, properly designed, scientifically engineered slope.
- To improve profitability of opencast mines.
- To enable Design-engineer/scientist develop suitable designs.
- As indicators that excessive steepening could lead to:

  * Slope failure, * Loss of production,

  * extra stripping costs to remove failed material, * closure of mine

Types of Rock Slope Failures

Failure in Earth and Rock mass

* Plane failure * Wedge failure * Circular failure * Toppling failure * Rock fall

Failure in earth, rock fill and spoil dumps and embankments

* Circular * Non-circular semi-infinite slope * Multiple block plane wedge

* Log spiral (bearing capacity of foundations) * Flow slides and Mud flow

* Cracking * Gulling * Erosion * Slide or Slump

The various design parameters of rock mass classification, such as, rock mass rating (RMR), slope mass rating (SMR) are used for pit slope design and support design.

The several modes of possible failures, such as, planar, wedge and circular failure have to be analysed. Computer software programme 'Galena' is predominantly used for analysis of different modes of failures —parameters, such as, slope angles and slope height are used to evolve safe and cost effective stripping ratio and optimum exploitation mechanisms.

The dip and the strike of discontinuities can be measured using Brunton Compass along the benches on the foot wall and hanging wall of the ore body. The orientation of the major plane of weakness with respect to pit excavation is a critical factor, which determines the slope stability. As long as the discontinuity planes are not mined out, there would be possibilities of sliding/failure to occur. The shear resistance of joint planes and factor of safety against sliding for slope must be very high. The factor of safety less than unity signifies a potential unstable zone. The stereo net analysis provides good presentation of structural data generated during the field mapping. The data on the joint orientation and the angle of internal friction is used as input for performing analysis.

In the majority of the cases, slope failure in rock mass are governed by joints that develop across surface form by one or several sets of joints.

To determine the safe optimum slope angle and introduction of intensive of slope movements, slope stability investigation need to be carried for stability analysis.

Factors that affect the stability of slopes are as below:

a) Properties of joint planes existing within the rock mass, i.e., orientation, continuity, frequency, strength of filling material, etc.

b) Strength of rock material

c) Water pressure within the rock

d) In situ stresses

e) Mining methods, i.e., extraction with the help of blasting or mechanical equipment.

Potential failures like toppling, planar, wedge is applicable to the jointed rock masses. In highly fractured or decompressed rock, circular failure may occur. Occurrences of any one fracture mode or combination thereof is controlled by the relative orientation of quarry face/dominant joint face, dominant joint sets, shear strength characteristics of joints, and their continuity/frequency. Ground water that influences the stability of quarry is responsible for major slope failures. Piezometer can be installed to measure the ground water pressure. The different types of failures are shown in Figure 5.1 (See Plate 1).

5.1 TYPES OF SLOPE FAILURES

5.1.1 Plane Failure

Simple plane failure is the easiest form of rock slope failure to analyse. It occurs when a discontinuity striking approximately parallel to the slope face and dipping at a lower angle intersects the slope face, enabling the material above the discontinuity to slide.

Plane failure occurred along the prevalent or continuous joints dipping towards the slope with strike near to parallel to the slope face. It occurs when a geological discontinuity such as bedding plane strikes parallel to the slope face and dip into the excavation at an angle greater than the angle of friction. It has been shown in Figure 5.2 (see Plate 1).

There are 2 unstable conditions that occur when critical joint dip is less than the slope and when the mobilised shear strength in the joint is not enough to ensure stability. Plane failure depends on joint continuity and also when the difference between dip direction of the slope and failed joints is less than 90^0.

5.1.2 Wedge Failure

Wedge failure occurs along two joints from different sets with intersecting dip towards slope. When two discontinuities strike obliquely across the slope face and their line of interaction daylights in the slope face, the wedge of rock resting on these discontinuities will slide down the line of interaction, provided that the inclination of this line is significantly greater than the angle of friction. It is depicted in Figure 5.3 (see Plate 2). A wedge failure depends on the geometry, multiple the joint mobilize shear strength. The size of the failure depends on the joint frequency and is usually minor as compared to plane failure.

5.1.3 Step-Path Failure

Step path failure is similar to plane shear failure, but the sliding is due to the combined mechanism of multiple discontinuities or the tensile failure of the intact rock connecting members of the master joint set.

5.1.4 Raveling

Weathering of material and expansion and contraction associated with freeze-thaw cycles are principle causes of raveling. This type of failure generally produces small rockfalls, not massive failures.

5.1.5 Toppling Failure

Toppling failure occurs along a prevalent and is continuous set of joint, which dip against the slope, with strike near parallel to slope face. Joint slips between them and is frequently weathered. In practice, two types of instability can exist, such as, minor toppling occur near the surface of the slope and dip toppling which can produce both determinations. In both the cases failure develops slowly. Surface toppling can cause rock falls but dip toppling seldom fail suddenly. The difference between the dip direction value of the slope and joints is more than 90^0. The details of failure modes are illustrated in Figure 5.4 (see Plate 2).

5.1.6 Circular Failure

- Circular failures generally occur in weak rock or soil slopes.
- Failures of this type do not necessarily occur along a purely circular arc, some form of curved failure surface is normally apparent.
- Circular shear failures are influenced by the size and mechanical properties of the particles in the soil or rock mass.

When the material is very weak as in tailing dump, in soil slope or when the rock slope or when the rock mass is very heavily jointed or broken, as in the waste rock dump, the failure is defined by a single discontinuity surface but will tend to follow the circular failure (see Figure 5.5).

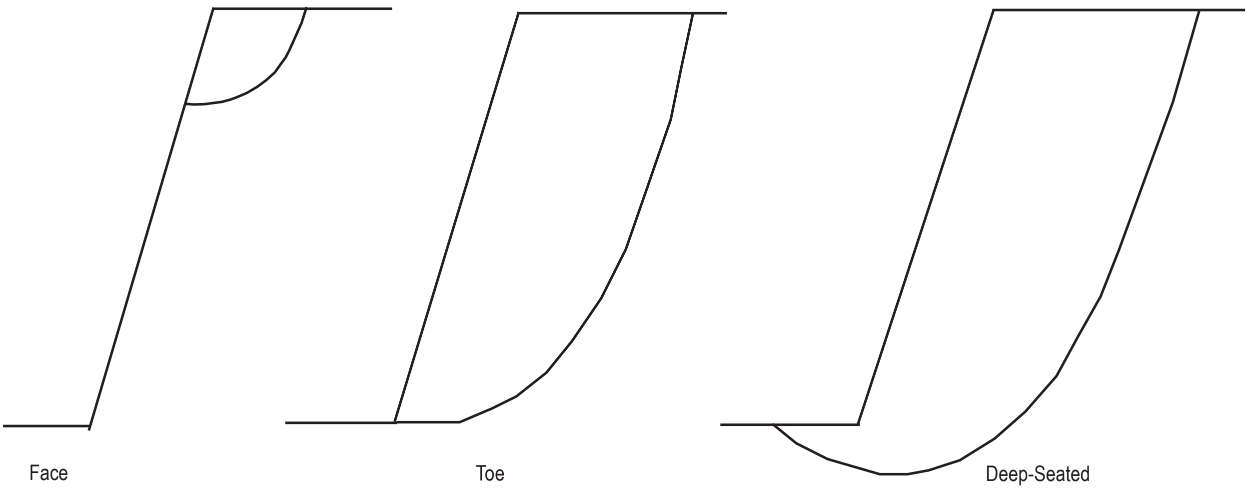


Figure 5.5: *Circular Failure*

Circular failure is classified into three types depending on the area that is affected by the failure surface. They are:

Slope Failure

In this type of failure, the arc of the rupture surface meets the slope above the toe of the slope. This happens when the slope angle is very high and the soil close to the toe possess high strength.

Toe Failure

In this type of failure, the arc of the rupture surface meets the slope at the toe.

Base Failure

In this type of failure, the arc of the failure passes below the toe and in to base of the slope. This happens when the slope angle is low and the soil below the base is softer and more plastic than the soil above the base.

5.2 GROUNDWATER FLOW

The incidence of slope failure in working mines during or shortly after periods of intense rainfall indicates the degree to which rainfall and subsequent movement of groundwater affect slope stability. A knowledge of groundwater conditions is needed for the analysis and design of slopes. The groundwater regime is often the only natural parameter that can be economically changed to increase the stability of slopes.

Water affects the stability of slopes in the following ways:

a) By generating pore pressure, both positive and negative, which alter stress conditions;

b) By changing the bulk density of the slope formimg material;

c) By both internal and external erosion; and

d) By changing the material constituents of the slope forming material.

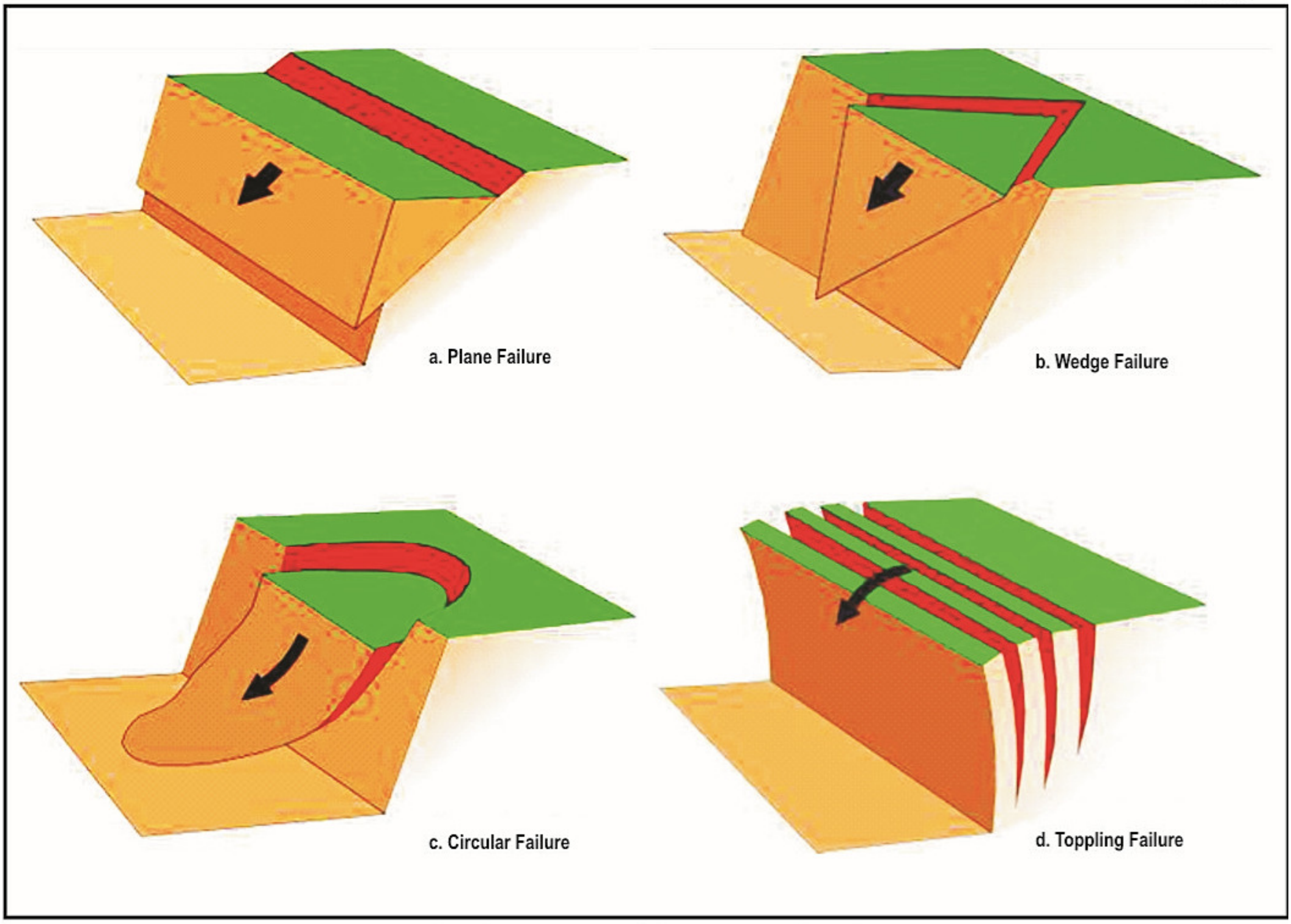


Figure 5.1 - Simplified Illustration of Most Common Slope Failure Modes

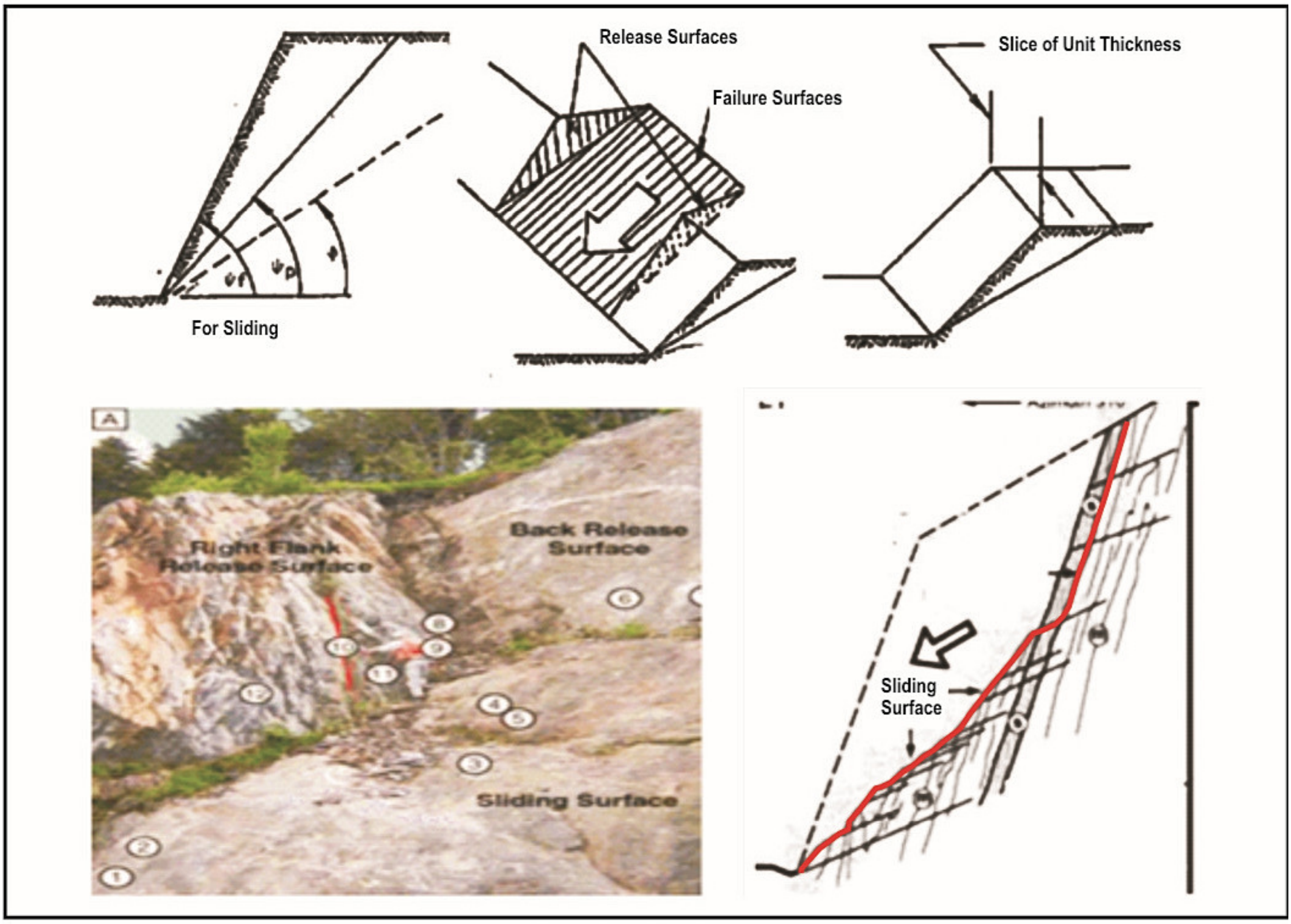


Figure 5.2 - Plane Failure

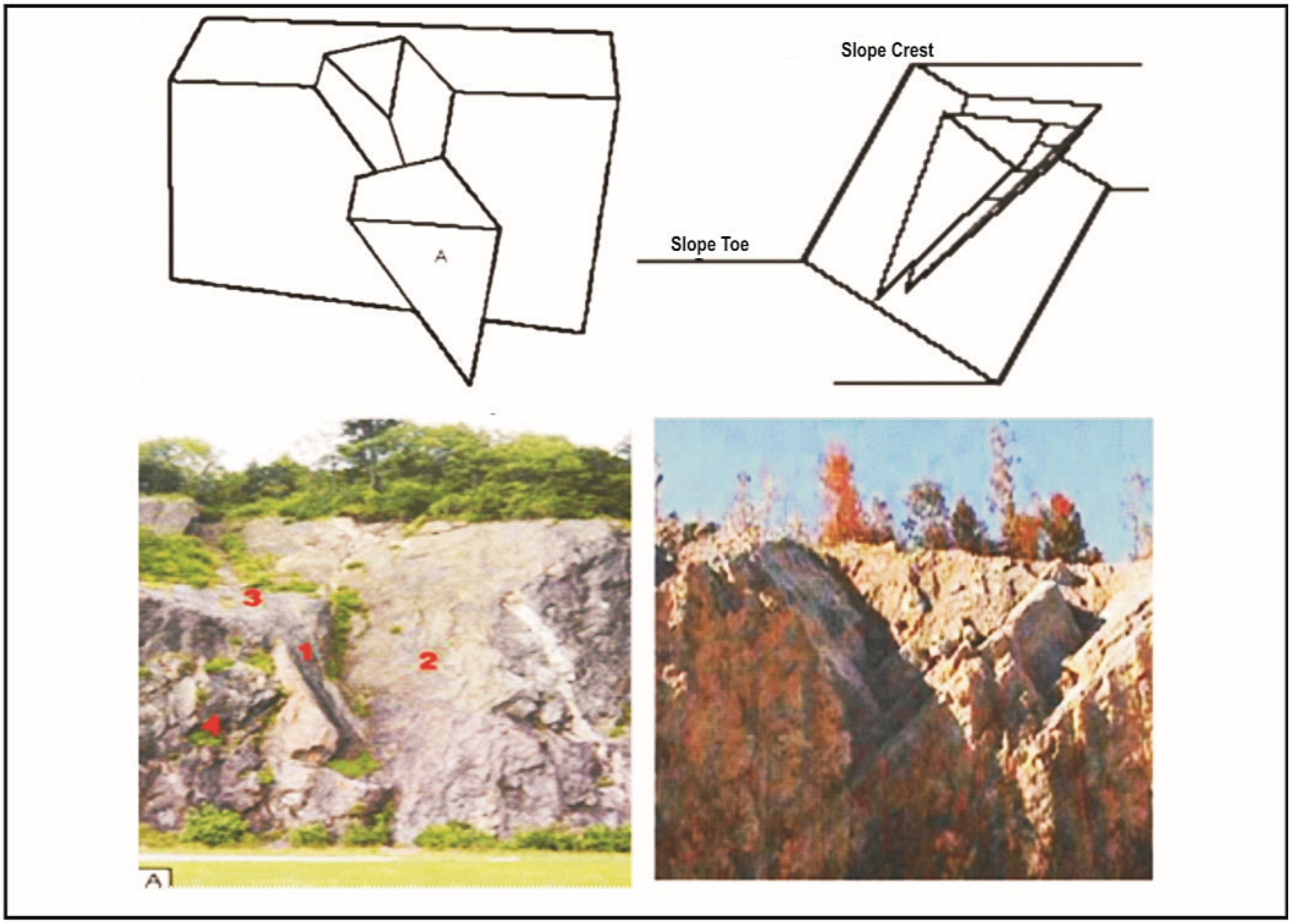


Figure 5.3 – Wedge Failure

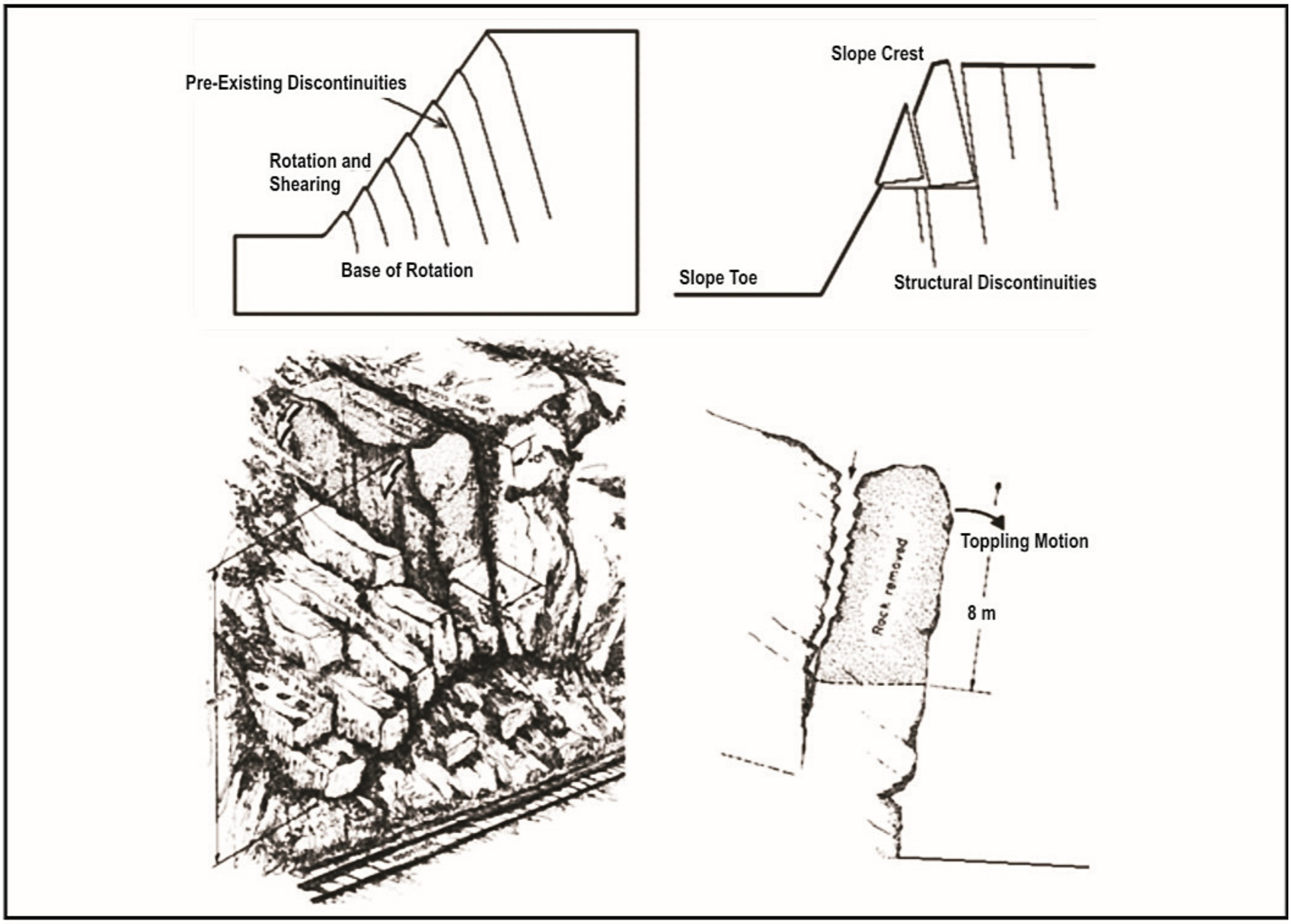


Figure 5.4 - Toppling Failure

5.2.1 Factors Affecting Slope Stability

Cohesion

It is the characteristic property of a rock or soil that measures how well it resists being deformed or broken by forces, such as, gravity. In soils/rocks true cohesion is caused by electrostatic forces in stiff over consolidated clays, cemented by Fe_2O_3, $CaCO_3$, NaCl, etc. and root cohesion.

However the apparent cohesion is caused by negative capillary pressure and pore pressure response during undrained loading. Slopes having rocks/soils with less cohesion tend to be less stable.

Angle of Internal Friction

Angle of internal friction is the angle (Ø), measured between the normal force (N) and resultant force (R), that is attained when failure just occurs in response to a shearing stress (S).

Its tangent (S/N) is the coefficient of sliding friction. It is a measure of the ability of a unit of rock or soil to withstand a shear stress. This is affected by particle roundness and particle size. Lower roundness or larger median particle size results in larger friction angle. It is also affected by quartz content.

Lithology

- The rock materials forming a pit slope determines the rock mass strength modified by discontinuities, faulting, folding, old workings and weathering.
- Low rock mass strength is characterised by circular raveling and rock fall instability like the formation of slope in massive sandstone restrict stability.
- Pit slopes having alluvium or weathered rocks at the surface have low shearing strength and the strength gets further reduced if water seepage takes place through them. These types of slopes must be flatter.

Ground Water

It causes the following:

- alters the cohesion and frictional parameters;
- reduce the normal effective stress;
- causes increased up thrust and driving water forces and has adverse effect on the stability of the slopes. Physical and chemical effect of pure water pressure in joints filling material can thus alter the cohesion and friction of the discontinuity surface.
- physical and the chemical effect of the water pressure in the pores of the rock cause a decrease in the compressive strength particularly where confining stress has been reduced.

Mining Method and Equipment

Generally there are four methods of advance in opencast mines. They are:

- strike cut – advancing down the dip
- strike cut – advancing up the dip
- dip cut – along the strike
- open-pit working

* The use of dip cuts with advance on the strike reduces the length and time that a face is exposed during excavation. Dip cuts with advance oblique to strike may often be used to reduce the strata
* Dip cut generally offers the most stable method of working but suffer from restricted production potential.
* Open-pit method are used in steeply dipping seams, due to the increased slope height are more prone to large slab/buckling modes of failure.
* Mining equipment which piles on the benches of the open-pit mine gives rise to the increase in surcharge which in turn increases the force which tends to pull the slope face downward and thus causing instability. Cases of circular failure in spoil dumps are more pronounced.

Slope Geometry

The factors effecting the slope geometry are given in Figure 5.6.

- The basic geometrical slope design parameters are height, overall slope angle and area of failure surface.
- With increase in height the slope stability decreases.
- The overall angle increases the possible extent of development of any failure to the rear of the crests and it should be considered so that the ground deformation at the mine peripheral area can be avoided.
- Generally overall slope angle of 45° is considered to be safe by Directorate General of Mines Safety (DGMS).
- Steeper and higher the height of slope less is the stability.

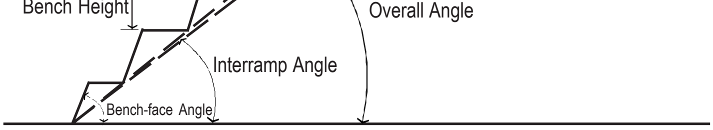


***Figure 5.6:** Typical Pit Slope Geometry*

5.2.2 Water Balance and the Hydrological Cycle

Water, whether in the solid , liquid or gaseous (vapour) form, is continuously in a state of circulation and transforms from one state to other by its movement between land, sea and air. This continuous movement is called the hydrological cycle.

In terms of the stability of slopes, the land-based portion of the hydrological cycle is most intense. Inflow to the system arrives as rainfall which can be extremely intense. Water flow from the system can be as run-off, evapotranspiration and subsurface underflow. Changes of storage within the system is that part of the rainfall which becomes incorporated into the groundwater system as recharge. The above elements form the basis of the water balance equation.

Rainfall = Evaporation + Run-off + Subsurface underflow + Change in soil moisture + Change in groundwater storage.

Changes in groundwater are critical to slope stability as it is these elements in the water balance equation that effectively alter the degree of saturation of the ground above the water table and the elevation of the water table itself.

Run-off is that proportion of rainfall that flows from a catchment into streams, lakes or the sea. It consists of surface run-off and groundwater run-off, where groundwater run-off is derived from rainfall that infiltrates into soil down to the water table and then percolates into stream channels. The amount of run-off in any given catchment depends on variety of factors, such as, the condition and nature of the soil and bedrock, the intensity and duration of rainfall, the slope angle, the surface of cover and the antecedent conditions within the catchment. The amount or depth of run-off maybe calculated by gauging the flow in streams which drain the catchment. The run-off coefficient or run-off percentage is defined as the proportion of rainfall that flows from a catchment as a percentage of the total depth of rainfall over the catchment area. Infiltration is defined as the movement of water from the ground surface into the soil or rock with the pores or interstices of the ground mass (i.e., the absorption of water by the soil).

Infiltration can be further divided into that part which contributes to the water content of the unsaturated zone, and the part which recharges the saturated groundwater system. Some recharge to the saturated groundwater system may be lost as groundwater run-off, whilst recharge to the unsaturated zone may be lost by transpiration or evaporation .When an unsaturated zone exists in a soil it is said to have a soil moisture deficit.

Recharge to this zone reduces the deficit until the soil becomes fully saturated, at which time the soil moisture deficit is equal to zero.

5.2.3 Types of Groundwater Flow

Water flows through soil or rock in various ways depending on the nature of the ground. Water-transmitting soil or rock units are called aquifers. Different types of aquifer demonstrate different modes of groundwater flow, such as, intergranular, fissure and conduit flow.

Intergranular flow is groundwater flow between the individual component grains that make up a soil or rock. This type of flow must closely follow the Darcy Concept of flow through an homogeneous isotropic medium of uniform grain size. In practice,however, most water-bearing strata exhibit intergranular or homogenous flow and path-preferential flow through fissures or conduits within the stratum. Joints within a soil/rock mass can have a significant effect on groundwater levels and hence contribute towards slope in stability.

An aquifer, therefore, can be simply defined as a permeable water-bearing stratum that transmits water under normal head or hydraulic gradient. An aquiclude is a stratum that may contain pore water but is not permeable enough to transmit water even under considerable hydraulic head. The term aquitard is used to indicate a stratum that shows limited water-transmitting capabilities.

Groundwater in aquifers not only exhibits intergranular and path-preferential flow characteristics related to the granular or fissured structure of the aquifer unit, but also exhibits confined or unconfined flow characterstics. In the former, groundwater is normally confined at the top by an impermeable stratum (aquiclude). The aquifer unit, therefore, gets fully saturated, and the piezometric (hydraulic) head is above the lower boundary of the confining medium. In unconfined flow situations, groundwater does not fully occupy the potential aquifer, and a free water surface (water table) exists within the aquifer.

Water table in aquifers can either be main table or perched water table. The main water table in any aquifer is the surface of the zone of complete saturation where water flows laterally under gravity, in the ground surface. Above the main water table, the soil or rock infiltration occurs through the soil or rock above the phreatic surface, and usually discharges to streams or the sea.

Perched water tables exist above the main water table where a localised reduction in basal permeability occurs in conjunction with recharge from above. Perched water tables may be transient, developing rapidly in response to heavy rainfall and dissipating quickly, or permanent, responding to seasonal variations in rainfall level.

5.3 NUMERICAL MODELLING

Numerical modelling techniques are practical and useful tool for maximising ore extraction. The most important aspect of this approach is to concert the model with observed and measured conditions based on the instrumentation. Geomechanical properties, such as, shear modulus, bulk modulus, density, cohesion of rock, friction of rock, cohesion of joints, joint friction angle, joint tensile strength are required for numerical modelling. Yield may occur in either solid or along the weak plane. The joint tensile strength along the joints may be assumed to be 0.01 MPa from engineering judgments. Modelling is conducted using two-dimensional microcomputer programme, FLAC (Fast Lagrangian Analysis of Continua). The result obtained by FLAC has to be checked by limit equilibrium method of analysis. FLAC is an explicit finite difference code to simulate the behaviour of rock and rock slope which may undergo plastic flow when their yield limit is reached. Finite difference technique is a complex computer technique for modelling a slope using small computerised geometric elements to which various material properties can be assigned. This method is capable of analysing the deformation and safety by computing stresses and strength of the slope. FLAC is particularly useful because it enables displacement to be modelled with time as excavation proceeds.

SNAP Computer Programme is used for the statistical analysis of the orientation data . It uses the lower hemisphere of polar equal area projection. SNAP is used to prepare density plot to delineate different joint sets present in the rock mass of the pits slope. Mean orientation of data, i.e., joint sets, dip direction, dip amount can be generated by this programme.

Limit equilibrium method can be used to verify the result obtained by FLAC. Plane failure, wedge failure, toppler failure, circular failure can be plotted on the hemisphere. Factor of safety for slope with water up to half of the depth of tension crack, factor of safety for drain slope with tension crack, factor of safety for drain slope without tension crack, factor of safety for slope with water up to half of the depth of slope but without tension crack could be represented. On the basis of different factors of safety for different conditions, one can judge optimum slope angle keeping in view the desired factor of safety.

5.3.1 Factor of Safety

Factor of safety (FOS) is an important tool to define the slope stability. Generally, the FOS is not constant but it is likely to be subjected to cyclic changes due to variations in water pressure, weathering and erosion of slope forming material. Further, factor of safety tends to decrease with time because of swelling of slope forming materials, effects of weathering, alterations in slope geometry etc. However, factor of safety can be enhanced by using different best practices during the mining operations. These practices include improvement in proper dewatering system to minimise the in situ moisture contacts of slope forming material which would consequently enhance the shear strength properties. In order to maintain safer slope, it is desirable to adopt slope-monitoring techniques.

5.3.2 Slope Monitoring

For the purpose of slope monitoring , the monitoring station can be located at the crest of the benches and by an electronic distance meter the precise level of any movement in and around the open pit could be determined.

5.3.3 Computer Programming

GALENA slope stability analysis programme can be used for modelling of slope stability. The input data include definition of axis limit, slope profile, material profile, pheratic and piezometric surface and method of analysis. The programme utilises limit equilibrium method of analysis to access factor of safety associated with slope with various configuration. The overall geology is itself in the definition of the model including the material properties. The slope profile can then cut through this model, as a slope would be excavated in real situation. Material above the slope profile is ignored since these will be mined out. In this way GALENA enables large number of analysis to be undertaken without redefining model each time.

While generating several models output fines from GALENA software, Bishops Circular Method of analysis can be performed. The selection of material properties for each rock type is an important input parameter. To define optimum pit slope angle, different sections of the hanging wall and footwall would need to be analysed .

The water level during the rainy season is also considered for the purpose of defining the pheratic surface. At certain reduced level, the pheratic surface modelling can be prepared. It is generally assumed that after certain reduced level, the saturated condition is likely to have its influence depending on the weathering of rock. Factor of safety should be computed from different sections. It may vary from 1 to 1.5 with certain slope angle and with increasing depth.

Sensitivity analysis could also be carried out to study the effect of factor of safety with respect to optimum slope angle and its corresponding stripping ratio. Based on the analysis, the proper slope angle could be achieved by considering favourable stripping ratio through proper mine planning, excavation sequencing by ground monitoring and proper support measures.

5.4 SLOPE STABILITY

The field of slope stability encompasses the analysis of static and dynamic stability of slopes of earth and rock-fill dams, slopes of other types of embankments, excavated slopes, and natural slopes in soil and soft rock. Slope stability investigation, analysis (including modelling) and design mitigation are typically completed by geologists, engineering geologists, or geotechnical engineers. Geologists and engineering geologists can also use their knowledge of earth process and their ability to interpret surficial geomorphology to determine relative slope stability based simply on site observations.

As seen in Figure 5.1 (see Plate 1), earthen slopes can develop a cut-spherical weakness area. The probability of this happening can be calculated in advance using a simple 2-D circular analysis package. A primary difficulty with analysis is locating the most-probable slip plane for any given situation. Many landslides have only been analysed after the fact. More recently slope stability radar technology has been employed, particularly in the Mining Industry, to gather real time data and assist in pro-actively determining the likelihood of slope failure.

5.4.1 Slope Stability Analyses

The slope stability analyses are performed to assess the safety of economic design of a human-made or natural slopes (e.g. embankments, road cuts, open-pit mining, excavations, landfills etc.) and the equilibrium conditions. The term slope stability may be defined as the resistance of inclined surface to failure by sliding or collapsing. The main objectives of slope stability analysis are finding endangered areas, investigation of potential failure mechanisms, determination of the slope sensitivity to different triggering mechanisms, designing of optimal slopes with regard to safety, reliability and economics, designing possible remedial measures, e.g. barriers and stabilisation mechanisms.

For successful design of slope knowledge in respect of geological information and site characteristics, e.g. properties of soil/rock mass, slope geometry, groundwater conditions, alternation of materials by faulting, joint or discontinuity systems, movements and tension in joints, earthquake activity etc. is vital. Choice of correct analysis technique depends on both site conditions and the potential mode of failure, with careful consideration being given to the varying strengths, weaknesses and limitations inherent in each methodology.

Before the computer age, stability analysis was performed graphically by using hand-held calculator. Today, engineers have a lot of possibilities to use analysis software that ranges from simple *limit equilibrium* techniques through computational limit analysis approaches (e.g. Finite element limit analysis, Discontinuity layout optimisation) to complex and sophisticated *numerical solutions* (finite-/distinct-element codes). It is must for an engineer to fully understand the limitations of each technique. For example, limit equilibrium is most commonly used and is a simple solution method, but it can become inadequate if the slope fails by complex mechanisms (e.g. internal deformation and brittle fracture, progressive creep, liquefaction of weaker soil layers, etc.). In these cases more sophisticated numerical modelling techniques should be utilised. In addition, the use of the risk assessment concept is increasing today. Risk assessment is concerned with both the consequence of slope failure and the probability of failure (both require an understanding of the failure mechanism).

In the last decade, development of Slope Stability Radar has enabled the engineers to remotely scan a rock slope to monitor the spatial deformation of the face. Small movements of a rough wall can be detected with sub-millimeter accuracy by using interferometry techniques.

5.4.2 Limit Equilibrium Analysis

The conventional limit equilibrium methods investigate the equilibrium of the soil mass tending to slide down under the influence of gravity. Transitional or rotational movement is considered on assumed or known potential slip surface below the soil or rock mass. In rock slope engineering, methods are highly significant for detection of simple block failure along distinct discontinuities. All methods are based on comparison of forces (moments or stresses) resisting instability of the mass and those that cause instability (disturbing forces). Two-dimensional sections are analysed assuming plain strain conditions. These methods assume that the shear strengths of the materials along the potential failure surface are governed by *linear* (Mohr-Coulomb) or *non-linear* relationships between shear strength and the normal stress on the failure surface analysis provides a factor of safety, defined as a ratio of available shear resistance (capacity) to that required for equilibrium. If the value of factor of safety is less than 1.0, slope is unstable. The most common limit equilibrium techniques are methods of slices where soil mass is discretised into vertical slices (Figure 5.7). Results (factor of safety) of particular methods can vary because methods differ in assumptions and satisfied equilibrium conditions.

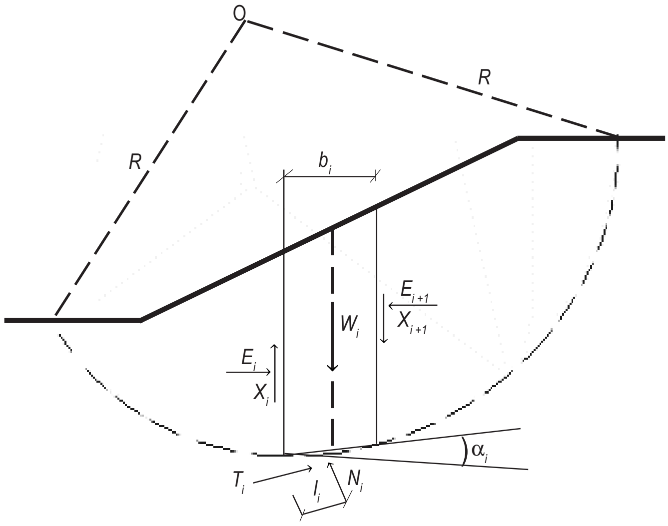


Figure 5.7: *Method of Slices*

Functional slope design considers calculation with the *critical* slip surface where there is the lowest value of factor of safety. Locating failure surface can be made with the help of computer programmes using search optimisation techniques. Wide variety of slope stability software using limit equilibrium concept is available including search of critical slip surface. The programme analyses the stability of generally layered soil slopes, mainly embankments, earth-cuts and anchored sheeting structures. Fast optimisation of circular and polygonal slip surfaces provide the lowest factor of safety. Earthquake effects, external loading, groundwater conditions, stabilisation forces (i.e., anchors, georeinforcements etc.) can also be included. The software uses solution according to various methods of slices (Fig. 5.7), such as, *Bishop simplified*, *Ordinary method of slices* (*Swedish circle method/Petterson/ Fellenius*), *Spencer*, *Sarma* etc.

Sarma and *Spencer* are called as rigorous methods because they satisfy all three conditions of equilibrium—force equilibrium in horizontal and vertical direction and moment equilibrium condition. Rigorous methods can provide more accurate results than non-rigorous methods. *Bishop simplified* or *Fellenius* are non-rigorous methods satisfying only some of the equilibrium conditions and making some simplifying assumptions.

Another limit equilibrium programme, SLIDE, provides 2D stability calculations in rocks or soils using these rigorous analysis methods—*Spencer*, *Morgenstern-Price/General limit equilibrium*; and non-rigorous methods—*Bishop simplified*, *Corps of Engineers*, *Janbu simplified/corrected*, *Lowe-Karafiath* and *Ordinary/Fellenius*. Searching of the critical slip surface is realised with the help of a *grid* or as a *slope search* in user-defined area. The Programme also includes probabilistic analysis using *Monte Carlo* or *Latin Hypercube simulation* techniques where any input parameter can be defined as a random variable. Probabilistic analysis determine the probability of failure and reliability index, which gives better representation of the level of safety. *Back analysis* serves for calculation of a reinforcement load with a given required factor of safety. The Programme also enables *finite element* groundwater seepage analysis.

The Program, SLOPE/W, is formulated in terms of moment and force equilibrium factor of safety equations. Limit equilibrium methods include *Morgenstern-Price*, *General limit equilibrium*, *Spencer*, *Bishop*, *Ordinary*, *Janbu* etc. This programme allows integration with other applications. For example, *finite element* computed stresses from SIGMA/W or QUAKE/W can be used to calculate stability factor by computing total shear resistance and mobilised shear stress along the entire slip surface. Then a local stability factor for each slice is obtained. Using a*Monte Carlo* approach, the programme computes the probability of failure in addition to the conventional factor of safety.

STABL WV is a limit equilibrium-based, Windows software based on the stable family of algorithms. It allows analysis using Bishop's, Spencer's and Janbu's method. Regular slopes as well as slopes with various types of inclusions could be analysed.

SV Slope is formulated in terms of moment and force equilibrium factor of safety equations. Limit equilibrium methods include *Morgenstern-Price*, *General limit equilibrium*, *Spencer*, *Bishop*, *Ordinary*, *Kulhawy* and others. This programme allows integration with other applications in the geotechnical software suite. For example, *finite element* computed stresses from SV Solid or pore-water pressures from SV Flux can be used to calculate the factor of safety by computing total shear resistance and mobilised shear stress along the entire slip surface. The software also utilises *Monte Carlo*, *Latin Hypercube* and the APEM probabilistic approaches. Spatial variability through random fields computation could also be included in the analysis.

Other programmes based on limit equilibrium concept are as follows:

- **GALENA** includes stability analysis, back analysis, and probability analysis, using the *Bishop*, *Spencer-Wright* and *Sarma* methods.

  GALENA is a simple, user-friendly yet very powerful slope stability software system that can simulate complex geological, groundwater and external force conditions without wrestling with frustrating computer instructions. GALENA has been developed by geotechnical engineers for practical use in the field. It was tested on a wide variety of earth and rock slopes, dams and cuttings. GALENA incorporates three methods of slope stability analysis which enable assessment of a wide range of ground stability problems in both soils and rocks.

- The BISHOP Simplified method determines the stability of circular failure surfaces.
- The SPENCER-WRIGHT method is used for either circular or non-circular failure surfaces.
- The SARMA method for problems where non-vertical slices are required or for more complex stability problems.

With all three methods available in one package, you can tackle any problem in a variety of ways without stopping to load a new programme.

- **GSLOPE** provides limit equilibrium slope stability analysis of existing natural slopes, unreinforced man-made slopes, or slopes with soil reinforcement, using *Bishop's Modified method* and *Janbu's Simplified method* applied to circular, composite or non-circular surfaces.

- **CLARA-W**, a three-dimensional slope stability programme includes calculation with the help of methods, such as, *Bishop simplified*, *Janbu simplified*, *Spencer* and *Morgenstern-Price*. Problem configurations can involve rotational or non-rotational sliding surfaces, ellipsoids, wedges, compound surfaces, fully specified surfaces and searches.
- **TSLOPE3**, a two- or three-dimensional programme analyses soil and rock slopes using Spencer method.
- **SLIDE**, is a slope stability programme developed by ROCSIENCE, Canada. An example of slope stability programme output in shown in Figure 5.8 (See Plate 3).

For Rock slope stability analysis based on limit equilibrium techniques, the following modes of failure need to be considered:

- *Planar failure* — case of rock mass sliding on a single surface (special case of general *wedge* type of failure); two-dimensional analysis may be used according to the concept of a block resisting on an inclined plane at limit equilibrium.
- *Polygonal failure* — sliding of a nature rock usually takes place on *polygonally-shaped* surfaces; calculation is based on a certain assumptions (e.g. sliding on a polygonal surface which is composed from *N* parts is kinematically possible only in case of development at least *(N - 1)* internal shear surfaces; rock mass is divided into blocks by internal shear surfaces; blocks are considered to be rigid; no tensile strength is permitted etc.)
- *Wedge failure* — three-dimensional analysis enables modelling of the wedge sliding on two planes in a direction along the line of intersection.
- *Toppling failure* — long thin rock columns formed by the steeply dipping discontinuities may rotate about a pivot point located at the lowest corner of the block; the sum of the moments causing toppling of a block (i.e. horizontal weight component of the block and the sum of the driving forces from adjacent blocks behind the block under consideration) is compared to the sum of the moments resisting toppling (i.e. vertical weight component of the block and the sum of the resisting forces from adjacent blocks in front of the block under consideration); toppling occur if driving moments exceed resisting moments.

5.4.3 Stereographic and Kinematic Analysis

Kinematic analysis examines which modes of failure can possibly occur in the rock mass. Analysis requires the detailed evaluation of rock mass structure and the geometry of existing discontinuities contributing to block instability. Stereographic representation (stereonets) of the planes and lines is used. Stereonets are useful for analysing discontinuous rock blocks. The Programme, DIPS, allows for visualisation structural data using stereonets, determination of the kinematic feasibility of rock mass and statistical analysis of the discontinuity properties. The Programme DIPS is available with ROC SCIENCE, Canada.

5.5 DETAILS OF ROCKFALL PROGRAMMES

5.5.1 Rockfall Simulators

Rock slope stability analysis may design protective measures near or around structures endangered by the falling blocks. Rockfall simulators determine travel paths and trajectories of unstable blocks separated from a rock slope face. Analytical solution method described by Hungr & Evans assumes rock block as a point with mass and velocity moving on a ballistic trajectory with regard to potential contact with slope surface. Calculation requires two restitution coefficients that depend on fragment shape, slope surface roughness, momentum and deformational properties and on the chance of certain conditions in a given impact.

The Programme, ROCFALL, provides a statistical analysis of trajectory of falling blocks. Method rely on velocity changes as a rock blocks roll, slide or bounce on various materials. Energy, velocity, bounce height and location of rock endpoints are determined and cuold be analysed statistically. The programme can assist in determining remedial measures by computing kinetic energy and location of impact on a barrier. This can help determine the capacity, size and location of barriers.

Various rock mass classification systems exist for the design of slopes and to assess the stability of slopes. The systems are based on empirical relations between rock mass parameters and various slope parameters, such as, height and slope dip.

5.5.2 Numerical Modelling

Numerical modelling techniques provide an approximate solution to problems which otherwise cannot be solved by conventional methods, e.g. complex geometry, material anisotropy, non-linear behaviour, in situ stresses. Numerical analysis allows for material deformation and failure, modelling of pore pressures, creep deformation, dynamic loading, assessing effects of parameter variations etc. However, numerical modelling is restricted by some limitations. For example, input parameters are not usually measured and availability of these data is generally poor. Analysis must be executed by well-trained user with good modelling practise. User also should be aware of boundary effects, meshing errors, hardware memory and time restrictions. Numerical methods used for slope stability analysis can be divided into three main groups—continuum, discontinuum and hybrid modelling.

5.5.3 Continuum Modelling

Modelling of the continuum is suitable for the analysis of soil slopes, massive intact rock or heavily jointed rock masses. This approach includes the *finite-difference* and *finite element* methods that discretise the whole mass to finite number of elements with the help of generated mesh (Figure 5.9 — See Plate 3). In *finite-difference* method (FDM), differential equilibrium equations (i.e., strain-displacement and stress-strain relations) are solved. *Whereas finite element* method (FEM) uses the approximations to the connectivity of elements, continuity of displacements and stresses between elements. Most of numerical codes allows modelling of discrete fractures, e.g. bedding planes, faults etc. Several constitutive models are usually available, e.g. elasticity, elasto-plasticity, strain-softening, elasto-viscoplasticity etc.

5.5.4 Discontinuum Modelling

Discontinuum approach is useful for rock slopes controlled by discontinuity behaviour. Rock mass is considered as an aggregation of distinct, interacting blocks subjected to external loads and assumed to undergo motion with time. This methodology is collectively called the*discrete-element* method (DEM). Discontinuum modelling allows for sliding between the blocks or particles. The DEM is based on solution of dynamic equation of equilibrium for each block repeatedly until the boundary conditions and laws of contact and motion are satisfied. Discontinuum modelling belongs to the most commonly applied numerical approach to rock slope analysis and the following variations of the DEM exist:

- distinct-element method
- discontinuous deformation analysis (DDA)
- particle flow codes

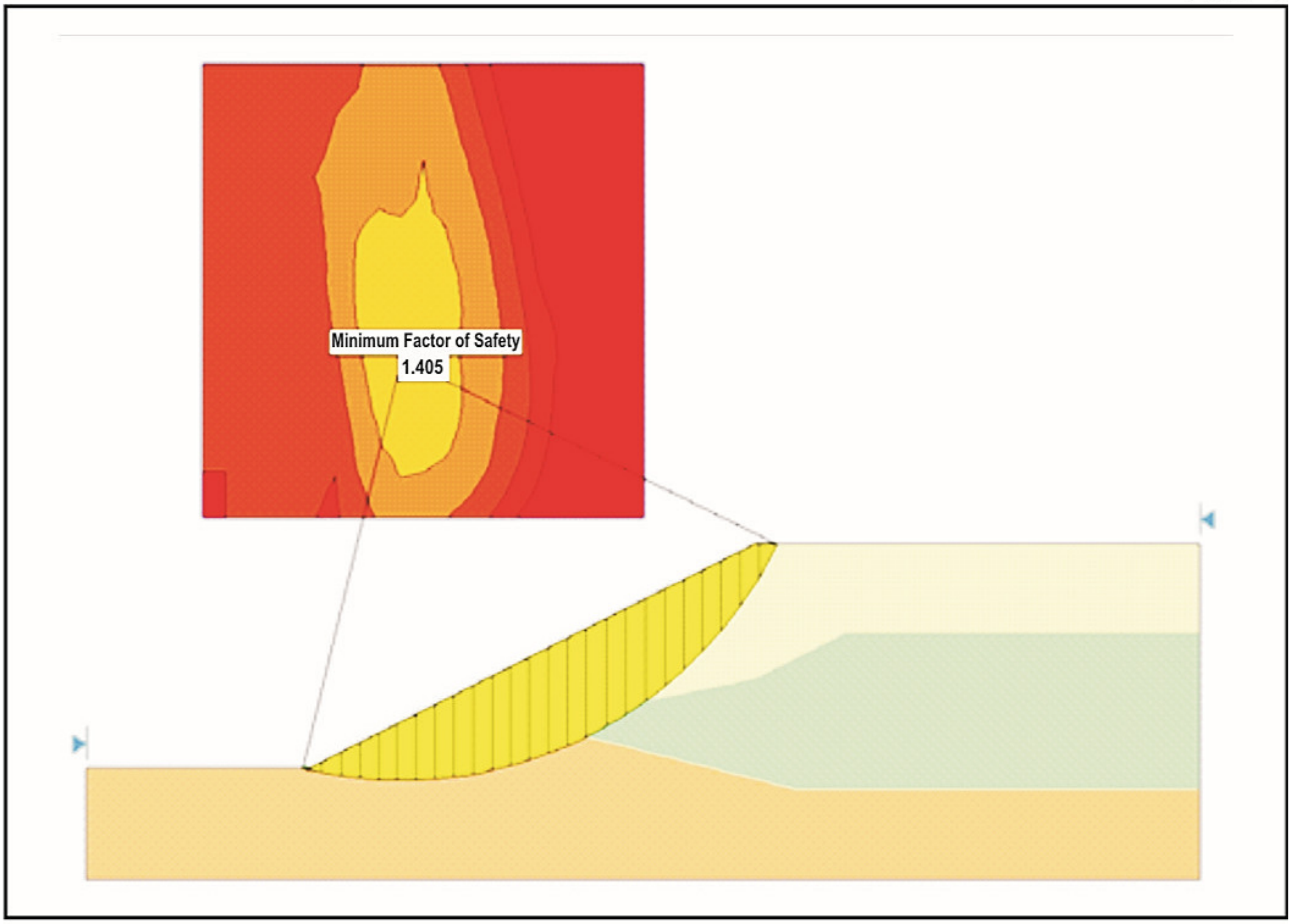


Figure 5.8 – Slope Stability Analysis Using SLIDE Software

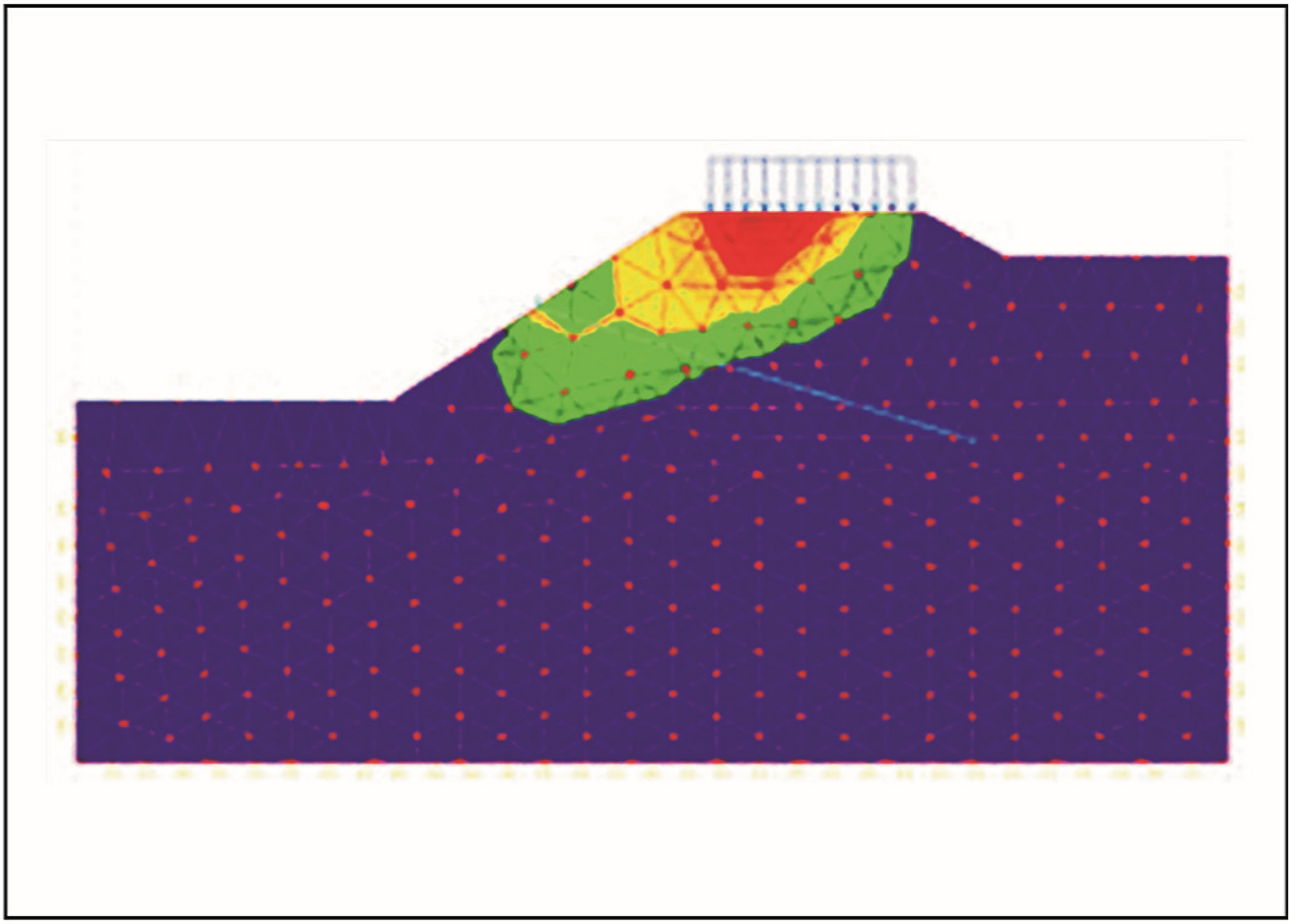


Figure 5.9 – Example of Finite Element Mesh

The *distinct-element* approach describes mechanical behaviour of both, the discontinuities and the solid material. This methodology is based on a force-displacement law (specifying the interaction between the deformable rock blocks) and a law of motion (determining displacements caused in the blocks by out-of-balance forces). Joints are treated as boundary conditions. Deformable blocks are discretised into internal constant-strain elements.

Discontinuum programme UDEC (Universal distinct element code) is suitable for high jointed rock slopes subjected to static or dynamic loading. Two-dimensional analysis of translational failure mechanism allows for simulating large displacements, modelling deformation or material yielding. Three-dimensional discontinuum code 3DEC contains modelling of multiple intersecting discontinuities and therefore it is suitable for analysis of wedge instabilities or influence of rock support (e.g. rockbolts, cables).

In *discontinuous deformation analysis* (DDA) displacements are unknowns and equilibrium equations are then solved analogous to *finite element* method. Each unit of *finite element* type mesh represents an isolated block bounded by discontinuities. Advantage of this methodology is possibility to model large deformations, rigid body movements, coupling or failure states between rock blocks.

Discontinuous rock mass can be modelled with the help of *distinct-element* methodology in the form of *particle flow* code, e.g. Programme PFC2D/3D. Spherical particles interact through frictional sliding contacts. Simulation of joint bounded blocks may be realised through specified bond strengths. Law of motion is repeatedly applied to each particle and force-displacement law to each contact. *Particle flow* methodology enables modelling of granular flow, fracture of intact rock, transitional block movements, dynamic response to blasting or seismicity, deformation between particles caused by shear or tensile forces. These codes also allow to model subsequent failure processes of rock slope, e.g. simulation of rock.

5.5.5 Hybrid/Coupled Modelling

Hybrid codes involve the coupling of various methodologies to maximise their key advantages, e.g. *limit equilibrium* analysis combined with*finite element* groundwater flow and stress analysis adopted in the SVOFFICE or GEO-STUDIO suites of software; coupled *particle flow* and *finite-difference* analyses used in PF3D and FLAC3D. Hybrid techniques allows investigation of piping slope failures and the influence of high groundwater pressures on the failure of weak rock slope. Coupled *finite-/distinct-element* codes, e.g. ELFEN, provide for the modelling of both intact rock behaviour and the development and behaviour of fractures.

5.6 TYPES OF INSTRUMENTS FOR MONITORING

Geo-technical instrumentation and monitoring plays an integral role in deriving meaningful conclusions related to strata behaviour in advance of mining or during mining operations. Geo-technical instrumentation is required for on-site observation and surveillance. Not all instruments are installed to monitor safety of a structure or construction operation or to confirm design assumptions but also to determine initial or background conditions.

Usually geo-technical instrumentation is carried out for taking measurements of strata behaviour and every measurement involves certain degree of error and uncertainty. Some instruments can have inherent defective designs, poor quality craftsmanship and materials to make the cost affordable. It is the experience and judgement that is important for tackling the shortcomings or gaps, which might influence the performance of the project.

Full benefit can be achieved only if every step in the instrumentation planning and execution phase is taken into consideration. Instrumentation schemes are normally adopted to serve one or more of the following purposes:

1. To determine the stability condition of mine structures
2. To check data assumptions
3. To generate data for future assumptions
4. To develop and evaluate new theories, concepts and guidelines
5. To settle legal disputes

Depending on the site conditions, the required number of types, capacity and least count of the instrument used should be decided. The range should be decided on the basis of the predicted behaviour of rock mass.

5.6.1 Instrumentation Types

Most geo-technical instruments consist of a transducer (a device that converts a physical change into a corresponding output signal), a data acquisition system and a communication system between these two. The basic instruments include Mechanical, Hydraulic, Pneumatic, Electrical and Electronic types. Sometimes, a combination of these basic types are also available depending on the application areas.

1. Mechanical Instruments

Dial indicators are used to convert the linear movement of a spring-loaded plunger to larger and more visible movement of a pointer that rotates above a dial. Dial indicator are more common, which can be used in mechanical crack gages, convergence gages, and mechanical tilt meters, fixed borehole extensometers, mechanical strain gages and mechanical load cells.

In the micrometer, rotation of finely threaded plunger causes the plunger to travel in or out of housing. Longitudinal movement of the plunger is measured, using a scale on the housing to indicate the number of revolutions of the plunger, frictional revolutions are determined using graduations marked around the plunger and a vernier on the housing.

2. Hydraulic Instruments

The two devices most frequently used to measure liquid pressure are Bourdon tube pressure gages and manometers. Bourdon tube pressure gages are more common and are used with hydraulic piezometers, hydraulic load cells, and borehole pressure cells and in some readout units for pneumatic transducers. Manometers are sometimes used with twin tube hydraulic piezometers and liquid levels settlement gages.

3. Pneumatic Instruments

Pneumatic transducers and data acquisition systems are used for pneumatic piezometers, earth pressure cells, load cells and liquid level settlement gages.

4. Electrical Instruments

Electrical resistance strain gages are used in many electrical instruments. Linear Variable Differential Transformer (LVDT) is used in fixed borehole extensometers and in other instruments for measurements of deformation. Director Current Differential Transformer (DCDT) has similar applications to LVDT and is usually preferred for geo-technical purposes. Linear potentiometers are an alternative to LVDT and DCDT for remote measurement of linear deformation. Rotary potentiometers are used for measurement of rotational deformation and where linear deformation can readily be converted to rotational deformation.

Variable Reluctance Transducers (VRT) are used in electrical crack gages and fixed embankments extensometers to measure linear deformation. Vibrating wire transducers are used in pressure sensors for piezometers, earth pressure cells and liquid level settlement gages, in numerous deformation gages, in load cells and directly as surface and embedded strain gages.

Force Balance Accelerometer is used as tilt sensors in tilt meters, inclinometers and in-place inclinometers. Similarly, magnet switch system is used in probe extensometers. Table 5.1 below illustrates general instrumentation types and their purpose of measurements.

Table 5.1: Instrument Types and Measurements

	Instruments	Measurements
1.	Piezometers	Pore water pressure and joint water pressure
2.	Earth pressure cells, Soil stress cell and Soil pressure cells	Total stress in soil
3.	Borehole pressure cell, Biaxial and Triaxial strain cells, Stress meters	Stress change in rock
4.	Surveying methods, Extensometers, Inclinometers, Transverse deformation gages	Deformation
5.	Load cells and strain gages	Load and strain in structural members
6.	Thermometer,Thermister, Thermocouple	Temperature
7.	Invar-wire extensometers, Borehole extensometer reading, multipoint borehole extensometer	displacement

Invar-wire extensometer as depicted in Figure 5.10 (a) & (b) [see Plate 4] is used for measuring the gross instability of benches. Multipoint borehole extensometer is used for measuring the displacement in strata during steepening process. The purpose of installation of extensometers is that it functions as a guide in respect of deducting the slope movements in advance of both steepening and providing cautions regarding the rock mass disturbances. Vibrating wire piezometers installed records the water level fluctuations in the strata. Similarly, multipoint borehole extensometers can be used for monitoring pit slopes as shown in Figure 5.11 (see Plate 5). Multipoint piezometers are used in boreholes for measuring water pressure and water level fluctuations as shown in Figure 5.12 (see Plate 5).

Borehole extensometer is used for monitoring the extent of movement inside the roof at different horizons. The instrument consists of spring anchors suitable for 43 mm diameter holes, steel wires and a reference collar. The anchors are made of spring steel and are fixed inside the borehole at desired depths. Stainless steel wire of 1 to 2 mm diameter is attached to each of the anchors. The free end of the steel wire is passed through a brass grip (button and screw arrangement), having identification numbers of the specific anchor . The "collar station" is a steel pipe provided with two screws or a male threading to facilitate the attachment of a portable readout unit. The steel wires from the individual anchors pass through the collar station and hang freely out of the borehole. The readout unit consists of a micrometer head and a dial gauge that serve both as tensioning and measuring device with accuracy close to 0.01 mm.

The displacement of the rock mass is regularly monitored by measuring the movement of the collar station with respect to each anchor. The instrument can have up to five anchors in a hole, and to a depth of 5 m and beyond. In case of bolted roof (ball length up to 1.8 m), the anchors could be installed in the roof—one within the bolted horizon and the second beyond the bolted horizon.

5.6.2 Site Selection

The instrument should be well-defined before taking up choice and type of instrumentation prior to installation at the site. The selection of instrument locations in the site should reflect predicted behaviour and be compatible with the method of analysis that will later be used when interpreting the data. Finite Element Analysis (EEA) is often used in identifying critical locations preferred instrument orientations and support measures. A practical approach for selecting instrument location involves the following considerations.

When selecting locations for monitoring, survivability of instruments should be considered and additional quantities should be selected to replace instruments that may become inoperative. Such locations should be selected so that data can be obtained as early as possible. There are several instruments available in the market but their selection is solely based on the type of data acquisition for which it is intended.

5.6.3 Geotechnical Instrumentation

During the slope steepening process, geo-technical instrumentation forms an integral part of the monitoring activity, which is essential for observing the behaviour of strata conditions. The slope-monitoring programme includes the following instrumentation.

a) Installation of multipoint borehole extensometer at different locations, in order to measure displacement in the strata during slope steepening process.

b) Installation of invar-wire extensometer/rock spy at related locations of the benches in order to measure the gross instability of the benches. The purpose of the installation of extensometers is to broadly guide the slope movements in advance of bench steepening and enable to forewarn the rock mass disturbance.

c) Multipoint borehole extensometers may be installed at selected locations to measure the relative displacement of the strata. For this purpose boreholes will be drilled and the anchors will be grouted at different depths. The relative movement of anchors is recorded with reference to standard reference point using LVDT and read-out unit.

The multipoint borehole extensometer consists of:

a) Head assembly used as reference point.

b) Standpipe to guide the extension rods.

c) Extension rods made of special alloy steel (8 mm).

d) Protective PVC rods for protection/isolation and free movement of extension rods.

e) Grouting anchors installed in drill hole in different depths for measuring relative displacement of strata enable measuring of displacement at three places, i.e., up to 6 m, 9 m and 12 m depths —three grouting anchors are placed within the borehole for the purpose.

f) Surface grouting pipe and air vent pipe.

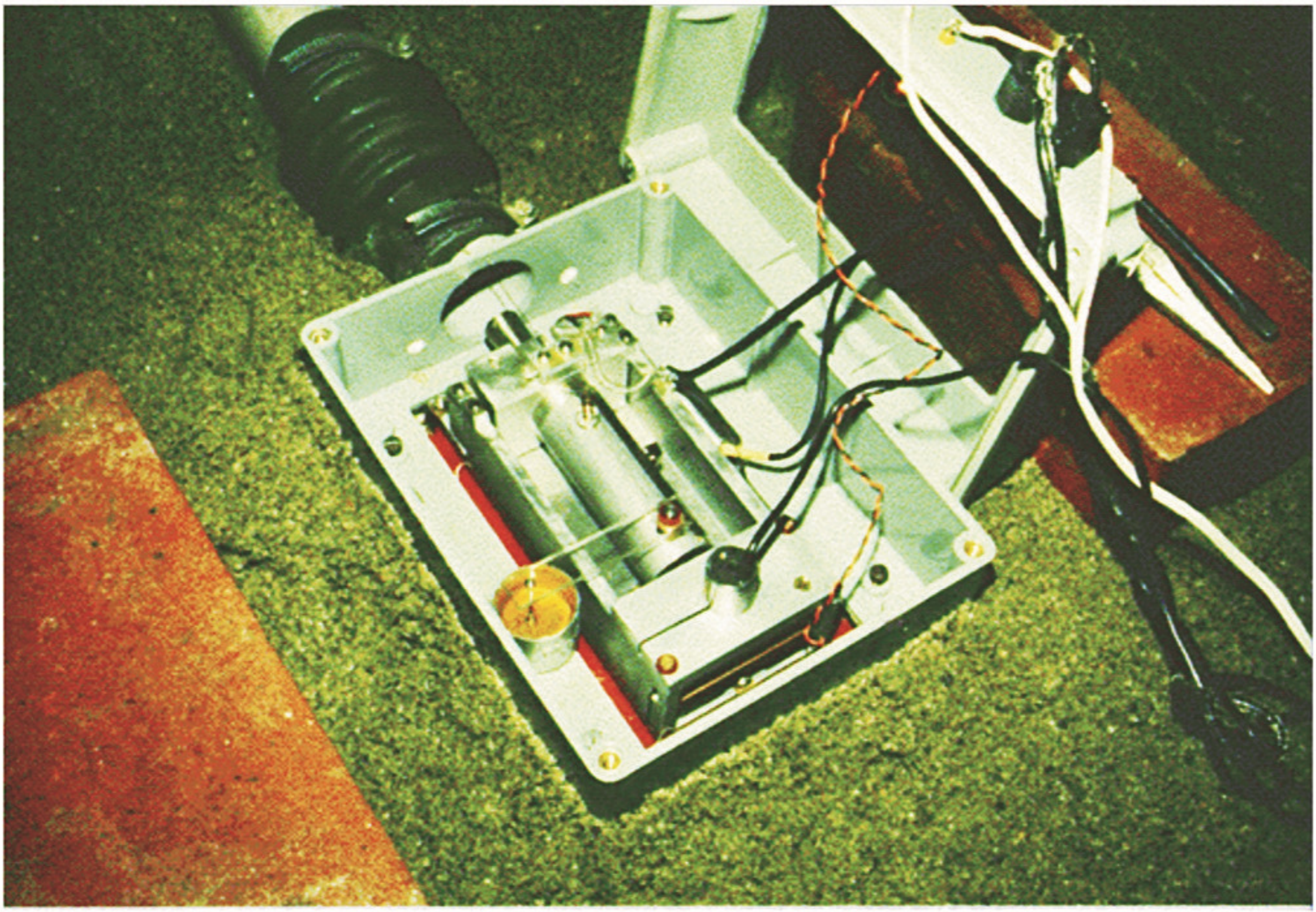


Figure 5.10 (a) - Invar Wire Extensometer

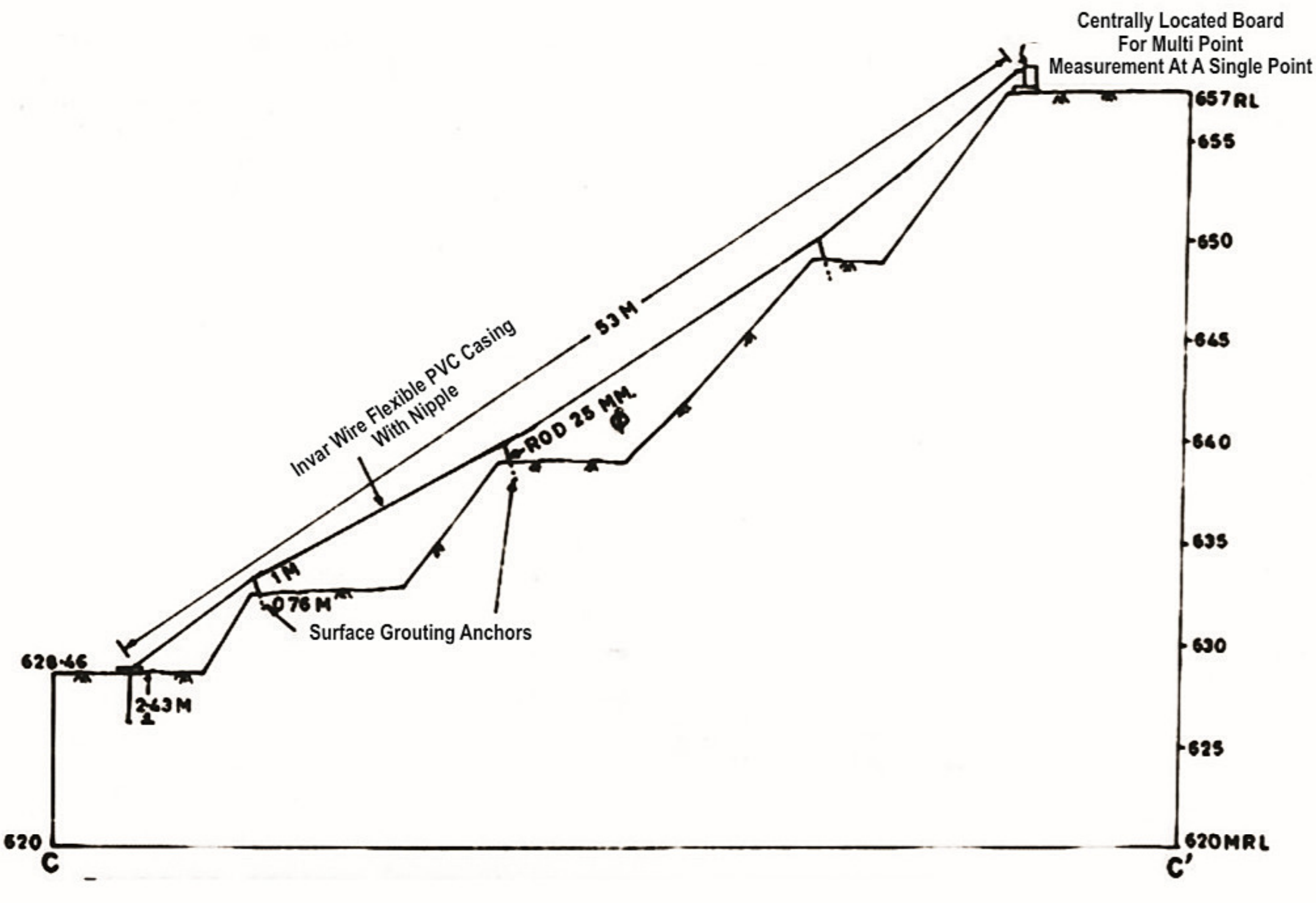


Figure 5.10 (b)– Monitoring Slope with Invar Wire Extensometer

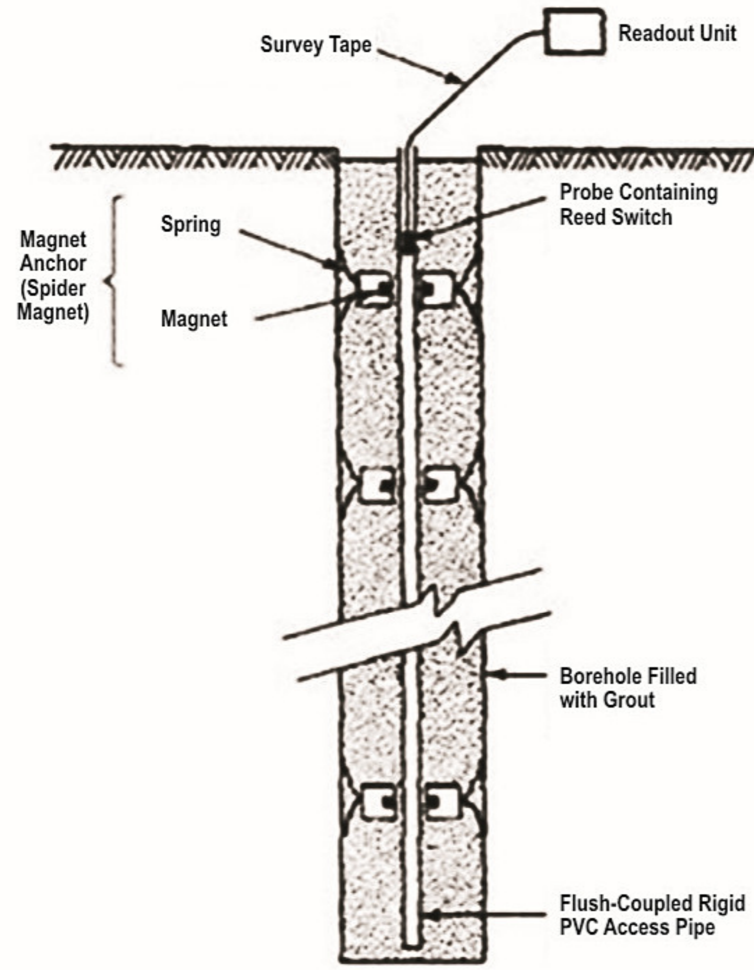


Figure 5.11 – Multipoint Extensometer

Figure 5.12 – Multipoint Borehole Piezometers

5.6.4 Monitoring Schedule

The installation and monitoring of extensometers will be carried out during all stages of slope steepening work.

a) Multipoint extensometers in the vertical boreholes drilled up to desired length. The purpose of these borehole extensometers is to measure the vertical displacement within the benches/strata.

b) Invar-wire extensometers may be installed at selected locations along the profile of the benches to measure the horizontal displacements, if any. This instrumentation also provides information on gross instability of slopes during slope steepening.

c) Rocks spy may be installed after observing the readings from multipoint borehole/invar-wire extensometers at a later stage, if required. Figure 5.13 (see Plate 6) indicates the instrumentation for monitoring of slope stability along the face.

5.6.5 Data Collection Procedures from Instrumentation Sites

The data collection from these instrument will be in the form of standard formats for each type of instrument. The limit for safe and unsafe conditions can be evolved after fixing the threshold limits based on field experience and engineering judgment. Apart from data analysis from instrumentation, it is recommended to have periodic site inspections by the mine personnel to observe any changes in the benches/strata.

5.7 SLOPE SUPPORT/STABILISATION

Bench support plan can be prepared based on the existing bench condition and geotechnical data generated. The types of supports were initially identified as

a) Systematic rock bolting/cable bolting with welded wire mesh

b) Cement grouting/shotcrete along with rock bolting

c) Constructional pack wall or buttresses

5.7.1 Mechanically Anchored Rock Bolts

The expansion shell anchored rock bolts of standard or bail type is the most common form of mechanical support. The expansion shell anchors operate basically in the same manner whether it is standard or bail type. A wedge attached to the bolt shank is pulled into conical expansion shell as the bolt is rotated. This forces the shell to expand against and into the wall of the boreholes. Borehole diameter is critical for installation of expansion shell rock bolts. To install the bolt, it is pushed into the hole until the preset torque is reached.

5.7.2 Grouted Rock Bolts

The commonly used grouted rebar or threaded bar is made up steel. Cement is used as grouting agent. The installation procedure is similar to mechanical type. The grout can be placed in the borehole either by pumping or by using cartridge. When the grout is pumped into the borehole the common practice is to push the grout tube into the bottom of the hole as tube is retrieved. It is important to ensure that the tube is in full contact with cement on it is retrieved to prevent the formation of air packets. When the hole is filled, the bolt is pushed into the hole through grout until it hit the length of hole or full contact is made between face plates and rock surface.

5.7.3 Rock Bolting with Grouting and Welded Wire Mesh

Before drilling, the bench face is to be dressed properly. Drilling should be carried out with the help of Jack hammer drill (21 mm) across the bench face. The hole should be drilled up to 3 m length. After drilling the hole either cable (standard wire rope, new or old) or torque steel rods (20 mm diameter) is inserted gently into the drill hole. Cement grout is then pumped into the hole with the help of grout pump. After setting of the mouth of the hole, face plate is tightened with the help of bolts. Welded wire mesh of 4 mm size with an aperture of 50 mm is generally used and the same will have to be tightened along with the grouted bolts.

5.7.4 Shotcreting

Shotcreting often in combinations with the bolting or anchoring is an effective method for preventing slaking and ravelling . It can be used for a wide range of strata conditions. It binds together loose blocks into coherent skin coal & seals each face to prevent wetting or drying. Before shotcreting the slope should be scaled to remove the loose bricks. There are two types of shotcretes, the dry-mix and the wet-mix. In dry-mix, water is added at the nozzle. Wet-mix shotcrete, basically contains the components of dry-mix along with water already added to the mixture. Dry shotcrete is widely used. The typical mix contains the following percentage of dry components by weight—cement 15-20%, Coarse aggregate 30-40%, Fine aggregates 40-50% and Accelerators 2.5%. The water to cement ratio (by weight) for dry-mix shotcrete lies in the range at 0.3 to 0.5 and can be adjusted as per site conditions. While applying the shotcretes, the nozzle should be placed as nearly perpendicular to the rock surface as possible and at a constant distance of about 1 m. Weep holes are drilled to relieve water pressure and horizontal pipes is installed behind the shotcrete to drain water from the permeable strata.

5.7.5 Rock Retention System (Pack Wall/Buttress)

Rock wall can be constructed as passive support measures. The construction includes providing proper foundation from the toe of the bench with the help of good quality concrete and raising the wall up to 1.5 m height. The void in between the benches and the wall should be packed with good quality material, such as, hard stone or rubble. This will arrest any damage to the concrete pillars up to a height of 1.2 m at regular intervals. The pillars can be connected with the help of three to four rows of connecting rods either welded or threaded. In between the bench face and the pillar it can be filled with waste rock or rubble which can act as a pack wall.

5.7.6 Support Design by Rock Wall

The main objective of the support design is to help the rock mass support itself. Pack wall with cement is constructed from the toe of the respective boreholes for providing support at weak/fractured/fragile zones. The construction of proper foundation at the toe of bench using good quality concrete and raising the wall up to 1.5 m height in stages till the entire bench height is reached is the method adopted for constructing pack wall. The void between the benches and the wall is packed with good quality hard stones or rubbles and weep holes are provided for proper drainage during rainy season in order to stabilise the upper boreholes. Figure 5.14 shows the typical pack wall constructed at different benches .

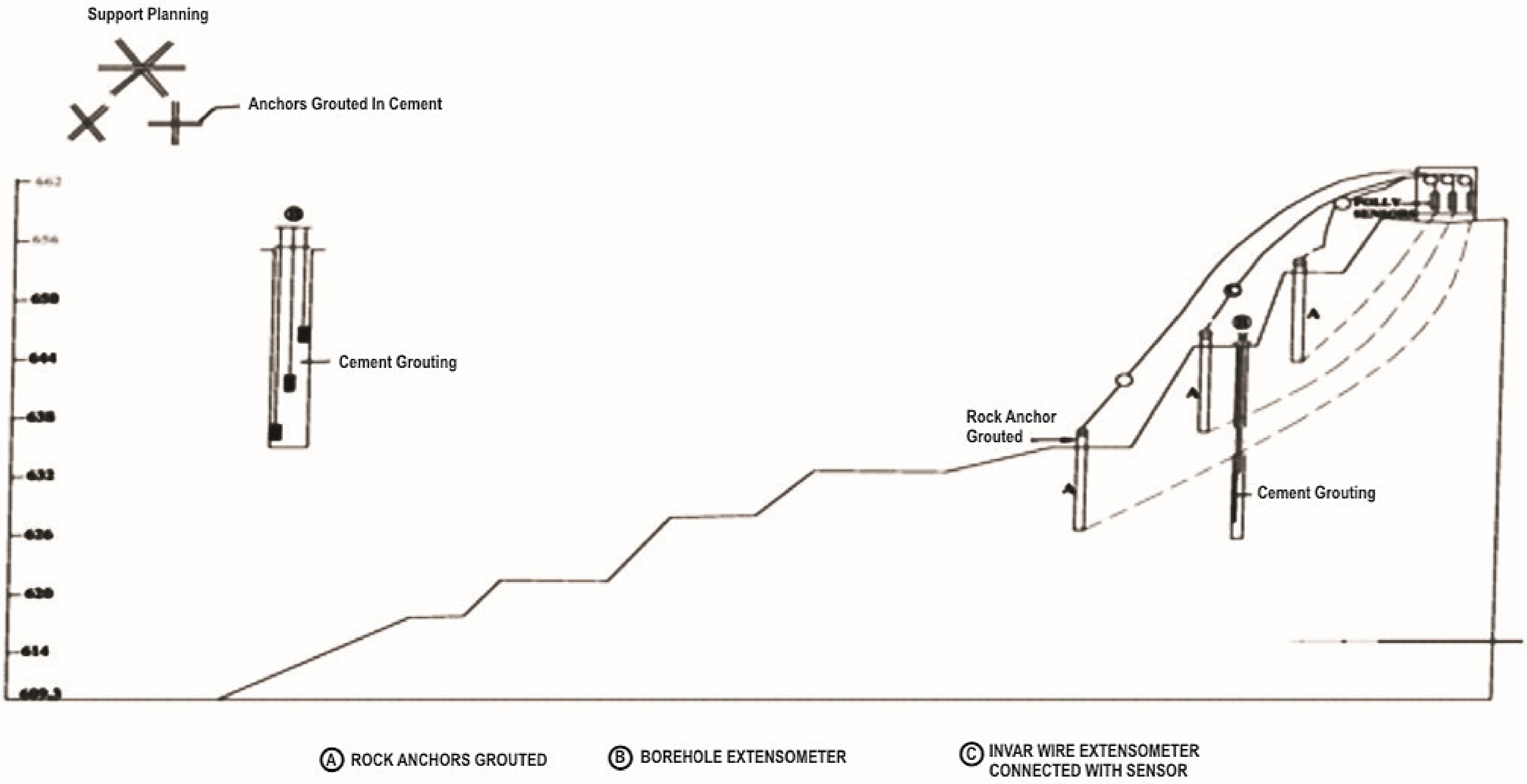


Figure 5.13 - Instrumentation for Monitoring of Slope Face Stability along the Face

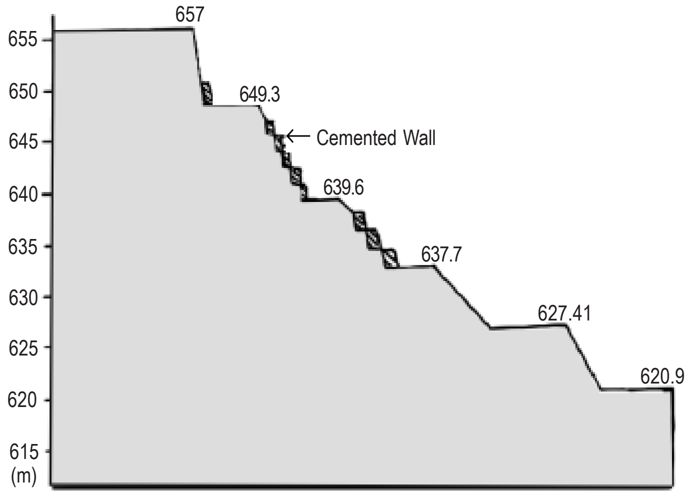


***Figure 5.14** : The Typical Pack Wall Constructed at Different Benches*

● ● ●

Chapter

6

Numerical Methods in Rock Excavation & Design

6.0 INTRODUCTION TO NUMERICAL METHODS IN ROCK MECHANICS

Several constitutive models have been included in the finite element method to describe different kinds of rock materials. In certain cases, the displacement models have also been adjusted to simulate the field conditions. New types of elements have been added to the basic codes to simulate joints and interfaces. Discrete element methods can also be developed to include non-linear behaviour by introducing piece-wise time-outing and the spring differences could be adjusted according to the applied loads.

In the boundary element methods, especially in displacement discontinuity analysis, various types of seam elements have been incorporated. The stress and strain behaviour of the seam elements can also be entered in digital fashion. However, the basic formulation still assumes a linear elastic homogeneous medium in which the rock is situated.

Recent advances in the field of numerical modelling have been the couplings of the different methods. Programmes which allow coupling enable better modelling of the field problems as they can consider both the infinite rock mass and the behaviour of the rock surrounding the location without losing accuracy or increasing the computational effort.

It is important to note that these methods yield qualitative results and trends and have to be interpreted in conjunction with field observation and instrumentation. The results are useful as guides and for evaluating options and for initial design in the case of complex problems.

6.1 NUMERICAL MODELLING

The representation of a problem in the form of mathematical equations and the solution of these equations to yield useful results is a broad definition of numerical modelling. With the advent of high-speed, low cost computers, the scope for the application of numerical modelling methods in engineering problems has increased thereby allowing it to almost replace empirical methods and thumb rules. The greater flexibility allowable due to the possible inclusion of a variety of influencing factors in the design process has made these methods popular. However, in most cases numerical modelling only helps in qualitative rather than quantitative analysis. This is especially true in the case of soil and rock mechanics as the geometry material properties and boundary conditions are not precisely known.

Several techniques have been developed for aiding the solution of ill-defined problems, such as, those encountered in soil and rock mechanics. Though they all have their origins in solid mechanics, modifications and additions through experience over time have led to the applicability of these techniques to design problems in mining and civil engineering.

6.1.1 Modelling Techniques

The analysis of stresses around mine openings is essentially a boundary value problem. Various techniques are available for the solution of these boundary value problems and they can be classed into two types—

1. Methods requiring the discretisation of the entire problem region.
2. Methods requiring the discretisation of the boundaries of the problem.

The effectiveness of mathematical modelling in rock mechanics depends on the diligence of the input data and on the degree to which the formulation is able to reflect the reality. In each of the above types of methods, the main procedure is as follows:

1. Define the geometry of the problem including all excavation and boundaries (natural and geological) that are expected to influence the problem.
2. Assign material properties to the different zones.
3. Define boundary conditions for the problem based on field observation/assumptions.
4. Apply the loading condition (either positive loading or excavation loading).
5. Determine the displacements due to the applied loads.
6. Determine the stresses resulting from these displacements.

The choice of continuum or discrete methods depends on many problem-specific factors, but mainly on the problem scale and fracture system geometry. Fig.6.1 (a-d) illustrates the alternative choices for different fracture circumstances in rock mechanics problems. Continuum approaches should be used for rock masses with no fractures or with many fractures, the behaviour of the latter should be established through equivalent properties established by a homogenisation process (Fig. 6.1a and d). The continuum approach can be used if only a few fractures are present and no fracture opening and no complete block detachment is possible (Fig. 6.1b). The discrete approach is most suitable for moderately fractured rock masses where the number of fractures are too large for continuum-with-fracture-elements approach, or where large-scale displacements of individual blocks are possible (Figure 6.1c).

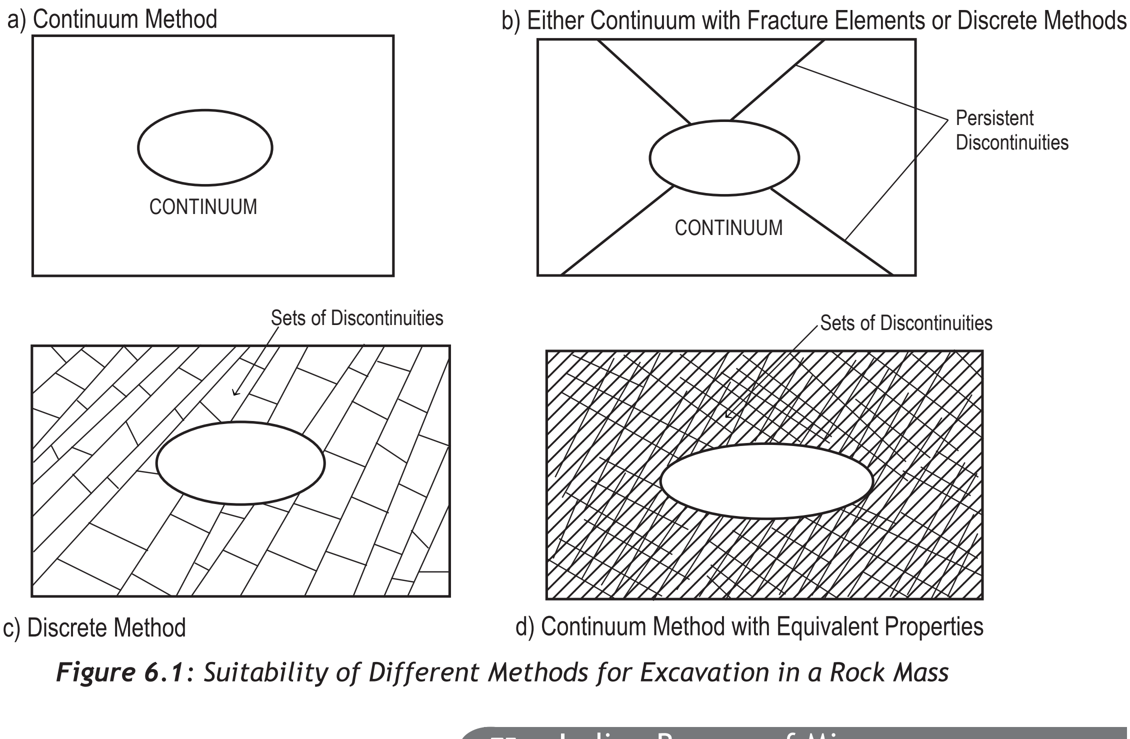


***Figure 6.1**: Suitability of Different Methods for Excavation in a Rock Mass*

Modelling fractured rocks demands high performance numerical methods and computer codes, especially regarding fracture representations, material heterogeneity and non-linearity, coupling with fluid flow and heat transfer and scale effects. It is often unnecessarily restrictive to use only one method, even less one code, to provide adequate representations for the most significant features and processes—hybrid models or multiple process codes are often used in combination in practice.

There are no absolute advantages of one method over another. However, some of the disadvantages inherent in one type can be avoided by combined continuum discrete models, termed hybrid models. In 1984, Lorig and Brady presented an early computational scheme in which the far-field rock is modelled as a transversely isotropic continuum using the BEM and the near-field rock as a set of discrete element blocks defined by rock fractures. This type of hybrid BED-DEM is shown in Figure 6.2. The complex rock mass behaviour caused by fractures and matrix, non-linearity in the near field of the excavation can be efficiently handled by the DEM or FEM, surrounded by a BEM representation of the far-field region with linear material behaviour without fractures.

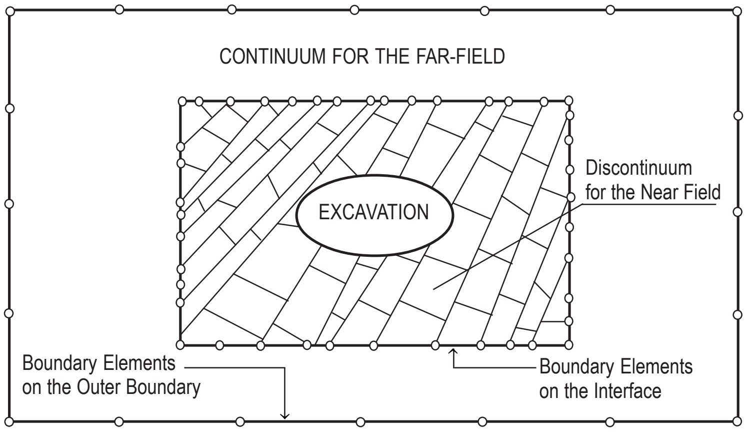


Figure 6.2*: Hybrid Model for a Rock Mass Containing an Excavation— Using the DEM for the Near-field Region Close to the Excavation and the BEM for the Far-field Region*

6.2 FINITE ELEMENT METHODS (FEM)

Finite element method is a tool for engineering analysis and offers a scope for application to a wide range of problems related to mining, mechanical, civil and other engineering disciplines. This method is now adopted in solving problems where complex structures, fluid dynamics, mine and tunnel structures and similar problems are to be addressed. Its application in mining engineering is now wide spread comprising of rock-support interaction, rock/dump slope stability analysis, tunnel and mine stability, drill bit with rock interaction, ventilation and others.

The basic concept in this approach is that a body or structure can be divided into a finite number of smaller units of finite dimensions called 'elements'. The original body or structure is then considered as an assemblage of these elements connected at finite number of joints called 'nodes' or 'nodal points'. The properties of these elements are formulated and combined to obtain the solution for the entire body or structure.

For structural analysis problems, the equations of equilibrium for the entire structure or body are obtained by combining the equilibrium equations of each element such that the continuity of the displacement is ensured at each node where the elements are connected. The necessary boundary conditions are then imposed and the equations of equilibrium are solved for the nodal displacement. Using these values of displacements at the nodes of each element, the strains and stresses are evaluated for each element using their properties.

Figures 6.3 and 6.4 (See Plate 7) show the division of two-dimensional meshed models, in which the modelled areas are divided into small triangular areas. This triangular area is called the element. The type of these elements varies depending upon the complexities of the problem, shape of the model area and solution requirements. In these figures the area is divided by 6-noded triangular element, which is mathematically stable and provides accurate results than other elements.

The finite element analysis has developed as an important and popular numerical method for engineering analysis and has undergone several modifications to suit the needs of the industries. It is applicable to a wide range of boundary value problems in engineering.

The following six steps summarise the finite element analysis procedure:

1. Discretisation of the Continuum

The continuum is the physical body, structure, or solid that is taken for analysis. Discretisation is the process in which the given body is subdivided into an equivalent system of finite elements. The finite elements may be triangles or quadrilaterals for a two-dimensional continuum, or tetrahedral, rectangular prisms or hexahedra in three-dimensional analysis. Although some efforts have been made to automate the process of subdivision, it is essentially a judgmental process to decide what number, size and arrangement of finite elements will give an effective representation of the given continuum.

2. Selection of Displacement Models

The assumed displacement functions or models represent only approximately the actual or exact distribution of the displacements. This is a basic approximation in the finite element method. The simplest displacement models are linear polynomials.

3. Derivation of the Element— Stiffness Matrix

The stiffness matrix consists of the coefficients of the equilibrium equations derived from the material and geometric properties of an element and obtained by the use of the principle of minimum potential energy. The stiffness relates the displacements at the nodal points to the applied nodal forces. The distributed forces applied to the structure are converted into equivalent concentrated forces at the node points. The equilibrium relation between the stiffness matrix (k), the nodal displacement vector (q) and the nodal force vector (Q) is expressed as a set of simultaneous linear algebraic equations.

$$(k)\,(q) = (Q)$$

The stiffness of a structure is an influence coefficient that gives the force at one point of the structure associated with a unit displacement of the same or a different point. The element of the stiffness matrix is called influence coefficients.

4. Assembly of the Algebraic Equations

The process includes the assembly of the overall or global stiffness matrix for the entire body from the individual element nodal force vectors by the direct stiffness method.

5. Solutions for the Unknown Displacement

The algebraic equations assembled in Step 4 are solved for the known displacements. In linear equilibrium problems, this is a relatively straightforward application of matrix algebra techniques. However, for nonlinear problems, the desired solutions are obtained by a sequence of steps, each step involving the modification of the stiffness matrix and/or load vector.

6. Computation of the Element —Stress and Strains

In certain cases the magnitudes of the primary unknowns, that is, the nodal displacements will be all that are required for an engineering solution. More often, however, other quantities derived from the primary unknowns, such as, strains and/or stresses, must be computed.

6.3 DISCRETE ELEMENT METHODS (DEM)

The discrete element methods allow finite displacements and rotations of discrete bodies, including complete detachment, and recognise new contacts between discontinuous objects (such as, rock blocks) automatically the calculation progresses. Of the various methods under this class, the distinct element method has been widely used in the analysis of jointed rocks.

The distinct element method is similar to the finite element method where the region of interest is divided into a system of solid elements. The method provides the capability of modelling jointed rock behaviour. This method is a relatively new technique that extends analysis capability beyond the finite element method. It is a computational technique developed specifically to model jointed rock mass behaviour. Originally it was developed to model jointed rock behaviour under low-stress situations where displacements due to joint movement far exceeded those of the intact rock. The method has now been extended to include highly deformable elements, with improved element-to-element contact laws and new contact detection logic. Also introduced were pore pressure effects and the ability of elements to break into two or more separate elements under certain combinations of contact loads and internal stresses.

The DEM is an explicit (time-marching) code that models an assembly of distinct elements of arbitrary shape. It employs a book-keeping scheme to store in memory movements and deformations of each element in an efficient manner and allows any element to interact with any other element in an arbitrary way. Elements may develop or break contact with other elements in any time step. This capability offers tremendous potential for simulating such failure modes as progressive vertical crack propagation associated with cutter roof failure, blocky roof failure, floor heave problems, caving behind a long wall face and subsidence.

The algorithm is based on a force displacement law and a law specifying the motion of each block due to unbalanced forces acting on it. In determining the forces mobilised by contact between blocks, a notional overlap, δn, is assumed to develop at block boundaries. In the present formulation, the normal force, Fn is computed using a linear force displacement law.

$Fn = Kn\ \delta n$, where Kn is the joint normal stiffness

The shear contact force depends on the deformation path to which the contact has been subjected. For an increment of shear displacement δs, the increment of shear force is given by

$Fs = Ks\quad \delta s$, where Ks is the joint shear stiffness

Plastic shear failure or slip occurs when

$(Fs) > \mu\ Fn$, where μ is the coefficient of friction

Shear and normal forces between the two blocks are set to zero when tension would occur.

The motion of an individual block is determined by the magnitude and direction of the resultant out-of-balance forces acting on each block. Numerical integration of the Newton's Second Law of Motion is used to determine the translation of the block centroid and the rotation of the block about the centroid.

Figure 6.3 - Finite Element Model of the Shrinkage Stope with Post pillar

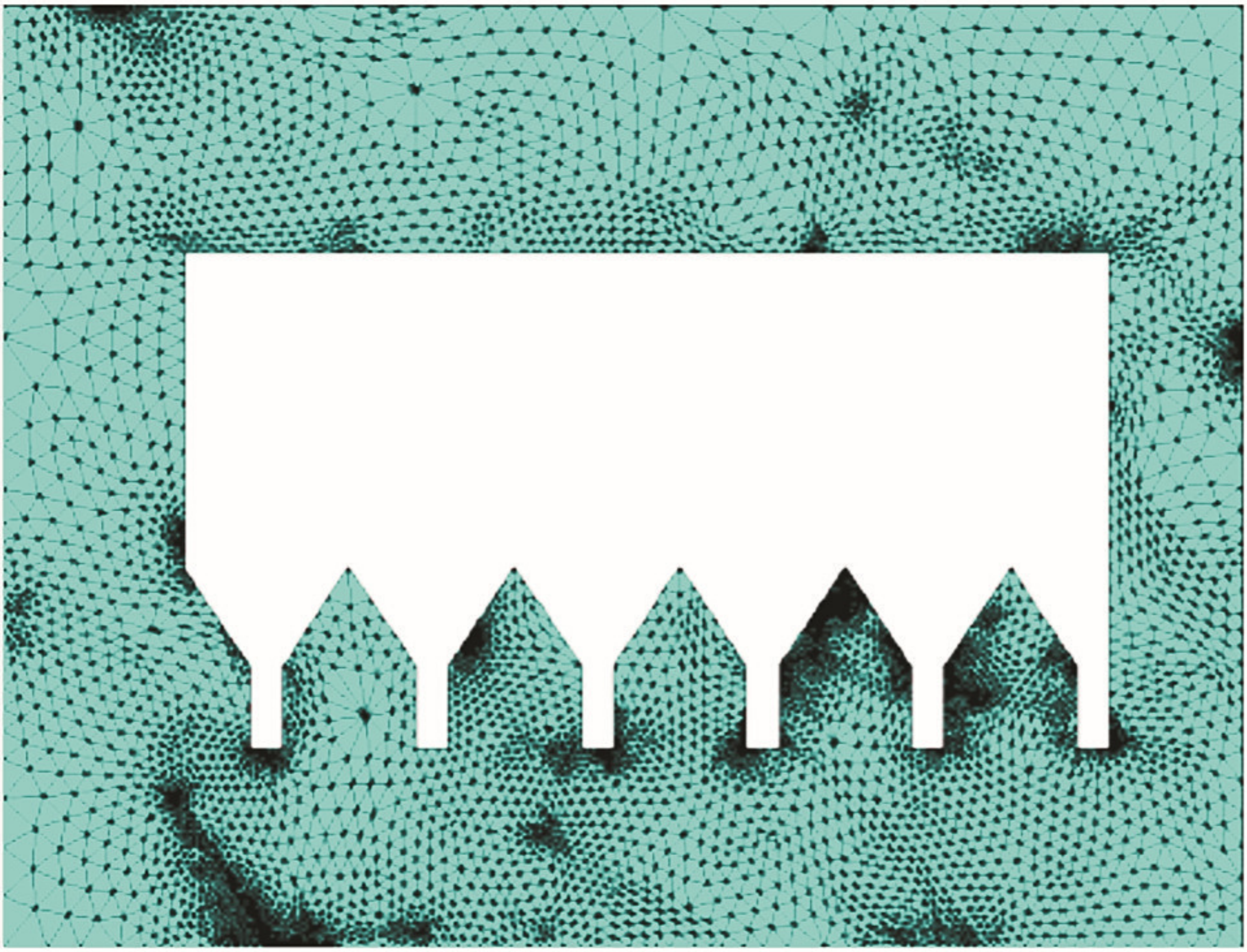


Figure 6.4 - Finite Element Model of the Shrinkage Stope without Post Pillar

Due to the explicit nature of the formulation, more complex constitutive joint behaviour can be simulated with little increase in the computational effort. The DEM is suited to model reinforced rock openings and to calculate their behaviour as the face is excavated. Rock bolts can be readily incorporated in the analysis. Progressive failure due to face advance and bolt reinforcement can be added as the excavation develops.

6.3.1 Boundary Discretisation Methods

In the above methods it has been seen that the whole region of the problem needs to be discretised. For the large problems encountered in rock mechanics, the computational effort required to obtain useful solutions will be very large. Boundary discretisation methods, which require less computation albeit with simplifying assumptions of a linear isotropic, homogenous and continuous elastic medium, have thus become quite popular for the solution of problems in rock and soil mechanics.

6.3.2 Boundary Element Method (BEM)

The boundary element method requires that only the boundary is discretised into elements. This method significantly reduces the programme input required compared with that for a finite element model of a similar problem. Thus the system of equations to be solved is much smaller.

6.4 APPLICATION OF NUMERICAL METHODS IN ROCK MECHANICS

There is no ideal numerical model for mining applications. The basic inputs and outputs from a numerical modelling procedure and their applications to rock mechanics are discussed below.

1. Numerical Models

A numerical model for mining application must accommodate the following:

- three dimensions
- any excavation shape
- a variety of material types
- failure of both plastic and time dependent nature
- structural discontinuities
- multiple excavation stages

The preparation of the input data and interpretation of outputs thus play a major role in the use of numerical methods in mining applications. Finite element formulations are generally more cumbersome and difficult to prepare and analyse. Boundary element methods are more popular due to ease of preparation of model. Discrete element methods are limited applications. Mixed formulations may provide a method to link up the various methods increasing the application of numerical methods to practical mine design.

Input data

The basic input data required by numerical modelling programme are

- The elastic properties of the rock mass
- The initial in situ stress state
- Rock mass strength properties
- Excavation shape
- The requisite boundary conditions
- The location of points in the rock mass where it is necessary to evaluate the stresses and displacements

Different programmes accept the above data in different format.

Elastic Properties of the Rock Mass

The programmes needs the values of the modulus of elasticity and the Poisson's ratio. In case of the non-linear elastic modelling problems, incremental values of the modulus of elasticity are required.

These material properties will be required for all litho units. However, if the difference in the properties is small or width of the litho unit negligible in comparison to a major litho unit, weighted average value can be assumed to reduce the number of material types. In the case of boundary element method, the rock medium is in any case assumed to be linear and elastic. In case the rock behaviour is time-dependent in nature, this must also be included in the equations that model the rock.

2. In situ State of Stress

In most cases, this information has not been determined in the field and simplified assumptions are used regarding the magnitude and direction of the in-situ stresses. The vertical stress and the horizontal stresses are assumed to be directly proportional to the depth below the surface.

3. Rock Mass Strength and Failure Criteria

Intact rock strengths are easily determined in the laboratory. The use of these values in modelling may lead to erroneous results, as the rock mass strengths may be considerably lower due to the presence of structural features, such as, joints, shears and faults. Rock mass strength is determined through large-scale field-testing, field evaluation or the application of the correction factors based on rock characterisation to the intact rock strength. Discrete element method requires information about the nature and orientation of joints, shears and faults for development of moving wedges.

The failure criteria define the relationship between the normal stress and the shear stress at failure. In many of the numerical methods, the rock can be defined as a no tension material and is assumed to have failed if the element is under tension. In case of shear failure, the modulus of the rock may change. The failure criterion is of importance in those methods where the evaluation is incremental and the model behaviour is affected in the next iteration due to the failure of some elements. In other cases, the model will show the nature of failure expected at critical location.

4. Excavation Shapes

Mine excavations are irregular in shape. For the purpose of modelling, the general shape of the excavation rather than the minor detail is of significance. Simplifying the assumptions about the shape can result in an easier and faster computation.

5. Boundary Conditions

As numerical modelling is essentially the solution of a boundary value problem, the selection of the requisite boundary conditions is an important part of the modelling process. Boundary conditions can be either stresses or displacements or both. It can be assumed that there will be no effect of the excavation on the displacements at a distance of three times the largest dimension of excavation. However, it is better to run several models including structural features and mine excavations at various distances to determine whether they have any influence on the displacements and stresses in the region of interest.

6. Location of the Field Points

The formulation should yield results, which are relevant to the problem being modelled and at points, which can be verified through field observations/measurements. The simplest measurement in the field is the measurement closure of the excavations and this can be easily related to the output from numerical modelling.

6.5 OUTPUT DATA

The output information is in the form of stresses and displacements at various points in the problem regime as desired by the user. This information is used initially to determine the validity of the model by crosschecking the results with field observations and instrumentation readings. The parameters of the model can be adjusted in such a way that the field observations are reflected in the model.

6.5.1 Sensitivity of the Model

The numerical modelling methods are useful mostly for qualitative analysis of field problems and attempts to relate the results to numerical values of strengths and instrument readings can be misleading. The purpose of such techniques is not to replace engineering judgment but to assist it in the evaluation of several options in the face of various influencing parameters. It is especially true as the input data used for the analysis are but idealisations of the actual field conditions.

6.5.2 Software for Numerical Modelling

Some of the numerical models for rock mechanics have been packaged and are commercially available to design-planning engineers. This software is mostly user-friendly and have preprocessor to make the preparation of input data easy. Powerful graphics post processor is used to display the results in a useful fashion.

1. BESOL Suite of Programmes

It is developed by M/s Geologic Research Inc., USA. BESOL is a versatile, easy-to-use system of computer programmes for solving rock mechanics problems in mining & geotechnical engineering. They are available in 2D and pseudo-3D suites and are based on the "fictitious stress method" and the displacement discontinuity methods" developed by Dr SL Crouch and AM Starfield. They are available with powerful pre- and post-processors and can run on desktop computers.

2. UDEC and 3DEC

These are design-and-analysis numerical programmes that enable studying the behaviour of jointed rock mass. Both are based on the distinct element method developed by Dr Peter Cundall and are available through M/s Itasca Consulting Group Inc., USA. They allow the far-field rock to be modelled with boundary elements and can model transient heat flow and thermally induced stress as well. The programmes are user-friendly and have powerful graphics post-processors and can be run on desktop computers.

3. FLAC

This is a powerful microcomputer programme that enable modelling of soil and rock behaviour. The programme is based on the finite difference method and can handle large distortion and non-linear materials. A large number of constitutive models are also available for modelling the rock behaviour. It is available through M/s Itasca Consulting Group Inc., USA.

4. BMINES

Programme BMINES is a static, two- and three-dimensional finite element computer programme used for the analysis of structural and geological systems. It is designed specifically for application to mining problems involving the simulation of excavation and construction sequences. The programme has both linear and nonlinear analysis capability. It also incorporates joint elements as well as rock bolt elements and it allows for prestressing of the structure prior to excavation. The programme also has mesh-generating facility. It is available from the US Bureau of Mines and can be run on Unix-based systems.

5. ADINA

The programme ADINA (Automatic Dynamic Incremental Non-linear Analysis) is a general purpose finite element programme used for static and dynamic displacement and stress analysis of structures with solids as well as fluids. It can perform linear and non-linear analysis in two and three dimensions. It has pre- and post-processing graphics facilities and is compatible with most mainframes and some mini-computers. It is available from the US Bureau of Mines.

6. NFOLD

The NFOLD programmes are displacement, discontinuity-based stress analysis programmes with extensive pre- and post-processing graphics facilities. The NFOLD suite has been developed from the basic algorithm of Dr SL Crouch with important modifications to include scaling and off-seam solutions. The programme is available from M/s Golder Associates (UK) Ltd, UK.

6.5.3 Determination of Optimal Orientation of Underground Mine Openings and Devising a Suitable Support System through Numerical Modelling

Software to be Utilised

The mine workings could be simulated by UDEC (Universal Distinct Element Code) developed by ITASCA, USA, UDEC is a two-dimensional numerical programme based on the distinct element method for discontinuum modelling. This software simulates the response of discontinuous media (such as, a jointed rock mass) subjected to either static or dynamic loading. The discontinuous medium is represented as an assemblage of discrete blocks. The discontinuities are treated as boundary conditions between blocks and large displacements along discontinuities and rotations of blocks are allowed. Individual blocks behave as either rigid or deformable material. Deformable blocks are subdivided into a mesh of finitely different elements, and each element responds according to a prescribed linear or non-linear stress-strain law. Linear or non-linear force-displacement relations for movement in both the normal and shear directions also govern the relative motion of the discontinuities. UDEC has several built-in material behaviour models, for both the intact blocks and the discontinuities, which permit the simulation of response representative of discontinuous geologic, or similar materials.

UDEC is based on a "Lagrangian" calculation scheme that is well-suited to model the large movements and deformations of a blocky system.

Input Parameters

The boundary conditions for modelling will be based on the measured in situ stresses of the particular deposit. The physical properties of coal and coal-measure rocks, viz., density, bulk modulus, shear modulus, cohesion & friction angle, tensile strength & compressive strength and joint properties will be taken into account.

Modus Operandi & Expected Results

Utilising the powerful software UDEC, the optimum direction of maximum principal stress can be decided. The choice of the optimal direction will be based on the minimum impact of adverse conditions, viz., stress around the opening & closure of the openings. The measurement of in situ stresses is not sufficient for design of an underground excavation. The effect of geological discontinuities (faults, dykes) creates perturbation in the stress. The optional direction of the excavation near the geological discontinuity may be different to the orientation of excavation in region far away from the discontinuity. Hence modelling will also be taken to analyse the effect of the geological discontinuities in the seam. Support systems will be designed to minimise the closure of the underground openings.

6.5.4 Computer Modelling and Software for Blast Design

Computer modelling of rock blasting continues to be an active research area. It holds the promise of predicting the results of a blast before it occurs, thus allowing blast to be simulated first on a computer before an expensive blast is attempted. The object of computer blast modelling is to apply the principles of physics and mechanics to a model geometry that represents the real geometry to be blasted. The application of mechanics makes the model dependent on measurable material properties. The models are developed with frequent comparison of computer simulations and field data. Some of the models are listed in Table 6.1. Use of these models on many real problems has shown many strengths and weaknesses.

Table 6.1: Computer Programmes for Blast Design and Analysis

Programmes	Developed by
3x3o – PRO	JKMRC, Australia (Cameron et at. 1991)
SABREX	ICI Explosives Group (Kirby et al. 1987)
BLASPA	Favreau (1980)
DMC	Sandia National Laboratory, USA (Taylor and Precce, 1989)
SOROBLAST	Lulea University, Sweden (Kou and Rustan, 1983)
DYNOVIEW & BLASTEC	Dyno Nobel, Inc. (Hopler, 1994)

Among these computer models, SABREX has been used in India (Bhushan and Srihari, 1990). This model requires rock properties, such as, density, compressive strength, tensile strength, Young's modulus, Poisson's Ratio, P- and S-wave velocities, crack attenuation factor and rock quality factor. Blast geometry is defined by blasthole diameter, bench height, hole depth, burden, spacing, drilling pattern and delay hook up. Explosive properties are calculated by a companion program called CPEX, a non-ideal detonation code. Given the chemical composition of the explosive, its density and detonation velocity at three different charge diameters, CPEX calculate explosives performance at any diameter and confinement. The essential properties of explosives used are density, detonation velocity, detonation pressure, explosion pressure, shock energy and three coefficients of pressure-volume curve. CPEX creates a file on calculated explosive, which can be directly assessed by SABREX.

A reference blast is first defined and all comparisons are made with respect to the blast. The cost is calculated on the basis of input geometry, unit cost of explosives, accessories and drilling. Two types of fragmentation calculations are carried out by SABREX—absolute and relative. Absolute fragmentation is in terms of size distribution of broken rock. Fragmentation at the toe level, along the charge column and in the stemming region is given in relative terms.

For relative comparison, a value of 100 is given for the reference blast. Any changes in fragmentation due to changes in geometry or charges are then calculated as relative number compared to the reference blast. A value less than 100 indicate poor breakage, and more than 100 means improved fragmentation.

Similar to fragmentation, throw is calculated in absolute and relative terms. For absolute prediction of throw, it requires additional inputs like angle of repose of the broken rock, number of rows blasted and delay between the rows. Given these inputs, velocity of rock movement is calculated. Fly rock prediction is only in relative terms. If the measured fragmentation and velocity of rock movement is available, the model can be calibrated and used as a tool for blast optimisation. The predicted results are displayed in colour graphs and in tables. Due to complexity of rock structure, difficulty in collecting data and a lack of understanding of the dynamic response of rock to the explosive loading, any predictions made by the model should be treated as guideline instead of an accurate prediction.

6.5.5 Blasting and Ground Vibration Study

When an explosive is detonated in the blasthole, it vibrates ground/ soil, particles with certain velocity and impart it into an acceleration. The most widely accepted measurement of ground vibration is peak particle velocity (ppv). The peak particle velocity is defined as the speed at which a particle of ground/ soil vibrates as the wave passes through a particular section.

Blast vibration study can be carried out with a view to establish the site specific propagation equation to know the maximum charge/delay in a round of blast to contain the vibrations within the permissible limits for known distances. Number of rounds of blast can be arranged . The vibrations can be measured at different structures, such as, temple, residential hutments, school, locosheds, explosives magazines etc. The distance from the place of blast can be measured. Explosive charge per round, diameter & depth of drill hole, type of explosives , number of detonators can be recorded. The site specific propagation equation is as below.

$$V = \alpha\left(\frac{D}{Q^{-0.5}}\right) - \beta$$

where, V = ground vibration (mm/s)

D = Distance from blasting site (m)

Q = Quantity of explosive (kg) fired instantaneously

α, β = constants determined based on data analysed

Ground vibration in terms of peak particle velocity for the structures in good condition and structures with visible damage should not exceed 10 mm/s and 5 mm/s. Accordingly, quantity of the explosive should be fired using above equation.

Such kind of study can also be utilised for calculating the percentage of fines produced during the blasting. Fragmented rock mass of each blast can be analysed using a screen of various mesh sizes. With different blasting parameters, quantity of the explosives to be used can be determined for desired fragmentation of the rock mass. Noise or air pressure can also be controlled using above pressure.

As per Director General of Mines Safety (DGMS), India's Technical Circular No.7 of 1997, the recommended permissible peak particle velocity (ppv) at the foundation level of structures in mining areas in mm/s is given in Table 6.2.

Table 6.2: Permissible Peak Particle Velocity (ppv) at the Foundation Level of Structures in Mining Areas in mm/s

No.	Type of Structure	Dominant Excitation Frequency Hz		
		<8	8–25	> 25 Hz
(A)	**Building/Structures not belong to the owner**			
(i)	Domestic houses/structures (Kuchha, brick and cement)	5	10	15
(ii)	Industrial buildings (RCC and framed structures)	10	20	25
(iii)	Objects of historical importance and sensitive structures	2	5	10
(B)	**Buildings belonging to owner with limited span of life**			
(i)	Domestic houses/structures (Kuchha, brick and cement)	10	15	25
(ii)	Industrial buildings (RCC and framed structures)	15	25	50

• • •

Chapter

7

Rock Mechanics Applications in Underground Mines

7.0 INTRODUCTION

One of the most important decisions facing a mine planner is the selection of a suitable mining method. Often the decision is made without a thorough knowledge of the ground conditions. Ignoring facts regarding ground conditions before endeavouring into any mining activity could lead to adverse situations.

It is a must that ground conditions be of requisite standards before undertaking open stoping or minimum support methods, such as, shrinkage stoping. On the other hand, poor quality ground is a necessity for block caving. The application of sublevel caving usually requires competent ore and incompetent easily caved host rock. The consequences of implementing block caving in competent, high strength rock with little or no fracturing are extremely coarse fragmentation, excessive drawpoint wear, high secondary blasting costs and, in general, unacceptable production costs. Conversely, the consequences of attempting sublevel caving of weak, highly fractured ground are poor brow control, ore loss, excess dilution, unnecessary blasting and, again, unacceptable production costs.

The choice of an underground mining method must be tailored to the ground conditions if the mining operation is to be successful. The emphasis should be put upon objectively assessing suitable mining methods and choosing the method most compatible with ground conditions.

7.1 BLOCK CAVING

Block caving is historically one of the lowest cost bulk underground mining methods. It is because of this, a detailed rock mechanics assessment of the ore body must be completed to determine its suitability.

The principal considerations in determining whether or not an ore body is suitable for block caving are—local geology, rock strength, hydrology, pre mine rock stress, ore body geometry and characteristics of the capping rock.

Depending upon the project scope and time constraints, a qualified mine engineer should be involved in order to gather and interpret the data used in making the final decision regarding the cavability of an ore body. Mine planning efforts for block caving projects should involve engineers with practical experience in the parameters that are under consideration for each particular project. Extensive experience in evaluating cut-off grades and ore reserves for potential block caving deposits along with level placement, type of caving most applicable, (i.e., gravity draw, slusher extraction or loader extraction) equipment, manpower, and cost estimation is required (refer Figure 7.1—Plate 8)

7.2 BLASTHOLE (SUBLEVEL) STOPING

This stoping method, along with its variations of sublevel stoping, vertical crater retreat (VCR) or end slicing, is especially suitable for ore bodies with the following characteristics:

a) The rock in the ore bodies and in the host ground is reasonably competent.

b) The ore zones have relatively large horizontal and vertical dimensions.

c) The number and size of barren or waste zones within the ore body is minimal.

When conditions are favourable for blasthole stoping, this method will produce reasonably low mining costs because it can be highly mechanised and result in good productivity per employee. Large diameter blastholes combined with modern diesel-powered load hauldump (LHD) equipment can usually be used to good advantage when this stoping method is employed. However, close supervision is required and an effective preventative maintenance programme must be enforced (refer Figure 7.2—Plate 8).

7.3 SHRINKAGE STOPING

Shrinkage stoping is the preferred method for mining steeply dipping relatively narrow vein deposits that have competent rock for the hanging wall and footwall. This method does, however, require that 60 to 70 percent of the broken ore be left in the stope until mining of the stope is completed, since the broken ore pile serves as the work platform for the miners. Consequently, the total revenue from the broken ore is delayed until sometime after each stope is completed. Also, ores that are susceptible to rapid oxidation upon exposure to air are generally not considered for shrinkage stoping.

Drilling and blasting is difficult to mechanise in shrinkage stopes because of the inherently uneven floors and generally limited working space. Drawing of the broken ore from the stopes can be mechanised using slushers or LHD equipment loading from drawpoints, or it may be done conventionally with chute loading of train cars or mine trucks. In general, shrinkage stoping is labor intensive and its use is limited to relatively small producers of higher grade ores (refer Figure 7.3—Plate 9).

7.4 CUT AND FILL STOPING

Cut and fill stoping (refer Figure 7.4—Plate 9) is an extremely versatile mining method that can be applied to both flat dipping and steeply dipping deposits under almost all types of ground conditions. It permits the mining of erratically-shaped ore bodies with a minimum of dilution, and a high degree of selectivity. Where ground conditions are reasonably good, an overhand extraction system is employed in which slices of ore are removed from a stope by starting at the bottom and advancing upward. Backfill is placed in the stope upon completion of each slice and serves as a working floor for extracting the succeeding slice. Where ground conditions are extremely poor in the ore body, an underhand extraction system is used in which work advances from the top of the orebody to bottom. In this case cement is added to the backfill material to stabilise it and provide a safe roof to work under.

Cut and fill stopes can often be highly mechanised so that employee productivity is good, but ore production from each stope must be periodically interrupted to allow for placement of backfill materials. A sufficient number of stopes must be made available so that mine production does not suffer because of the backfilling operations. The backfill material usually consists of deslimed concentrator tailings, and may be augmented by waste muck from mine development or by surface sand and gravel. Cement is commonly added to the backfill material to help stabilise it, either to provide a sound working floor

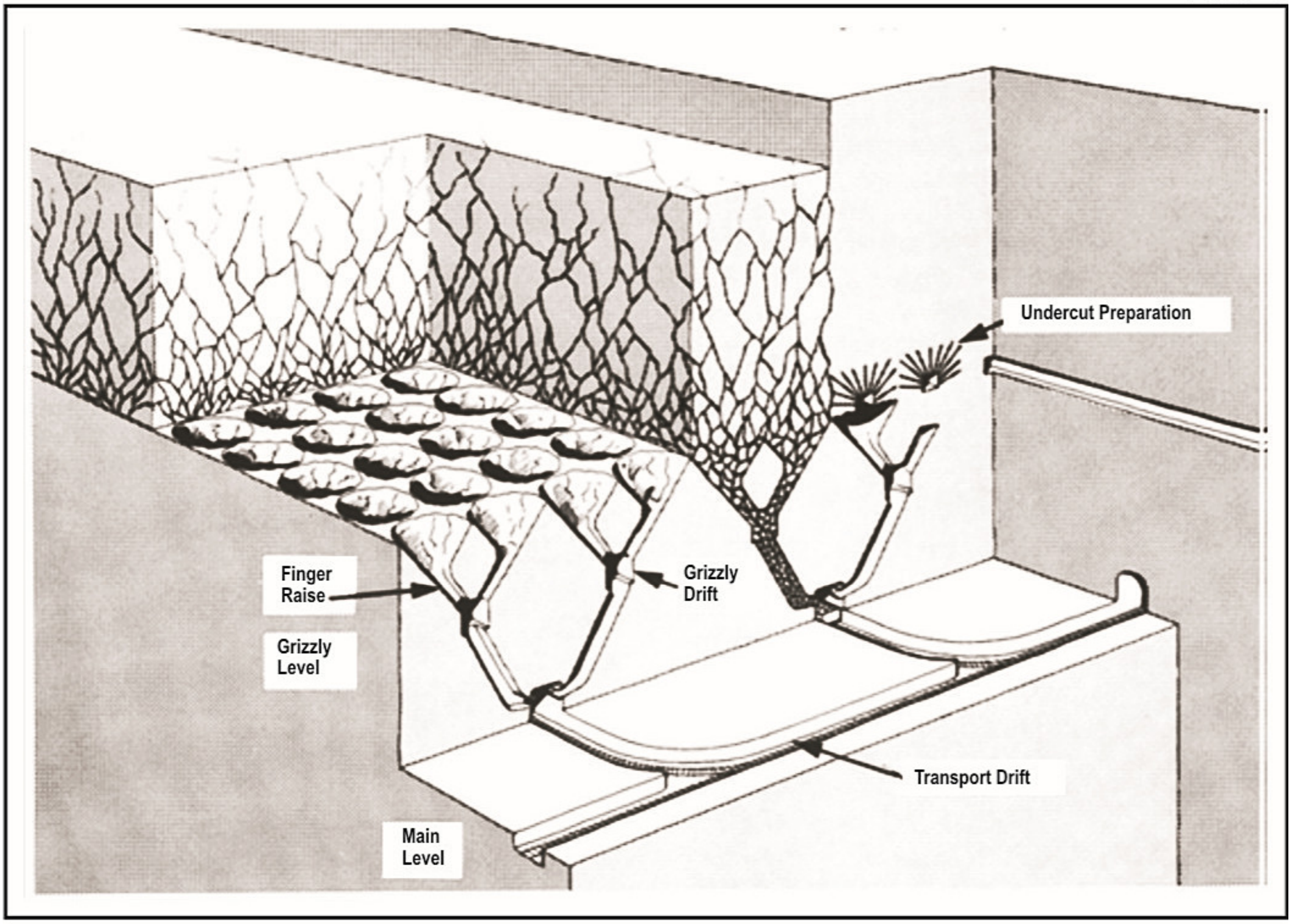


Figure 7.1 - Block Caving

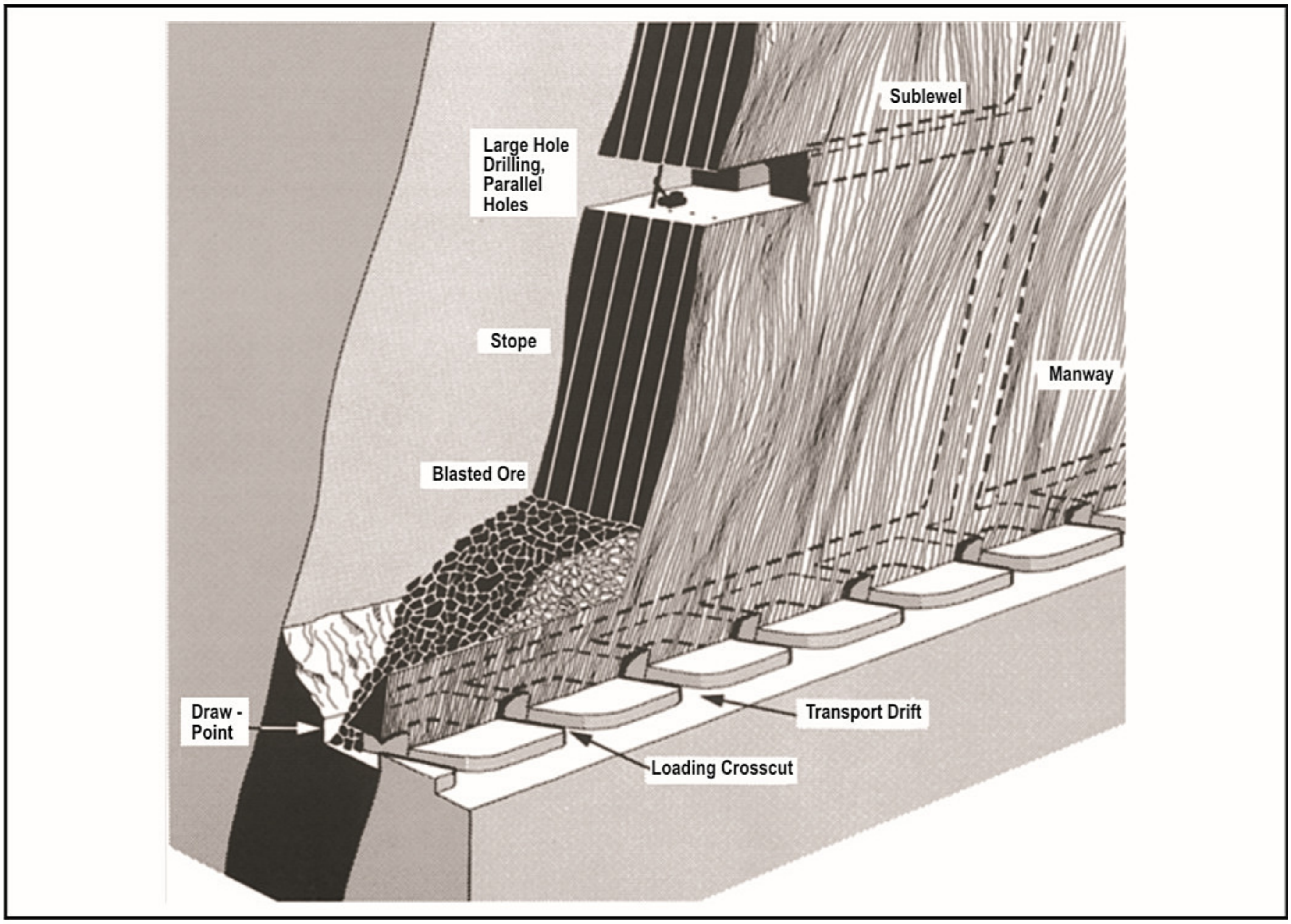


Figure 7.2 - Blasthole (Sublevel) Stoping

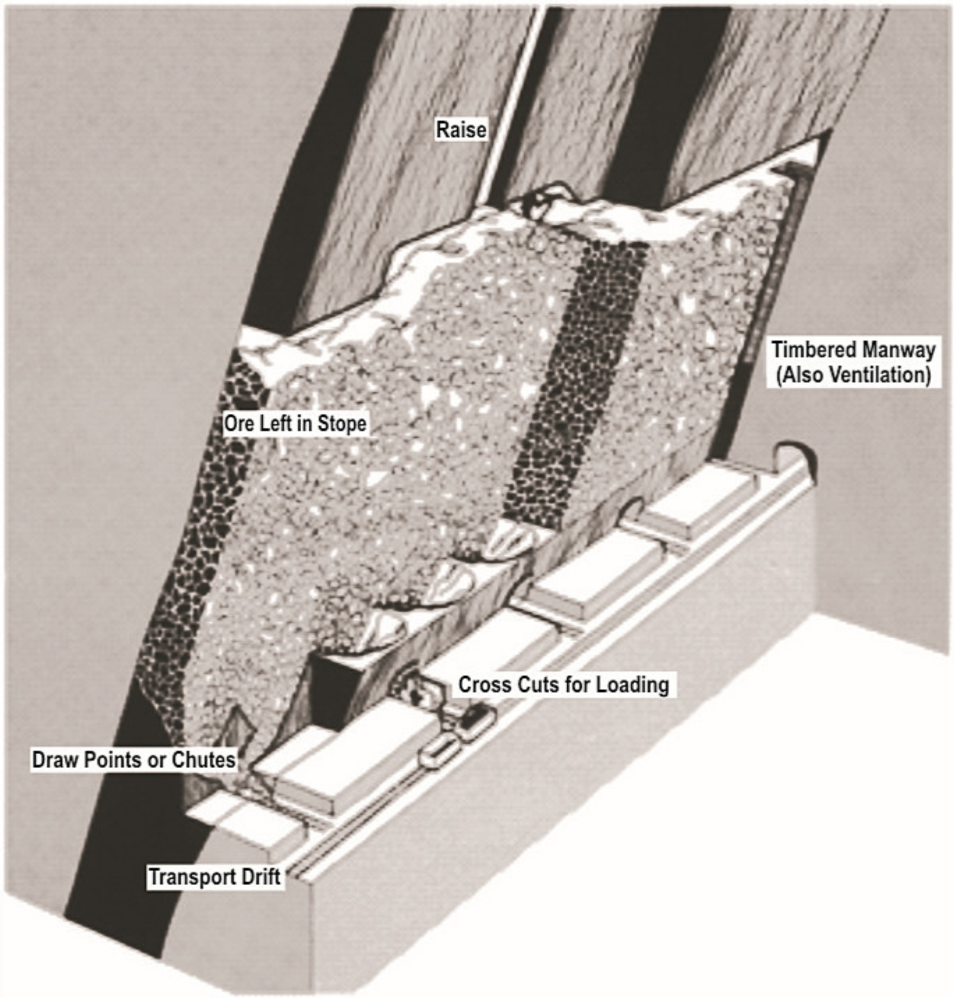


Figure 7.3 - Shrinkage Stoping

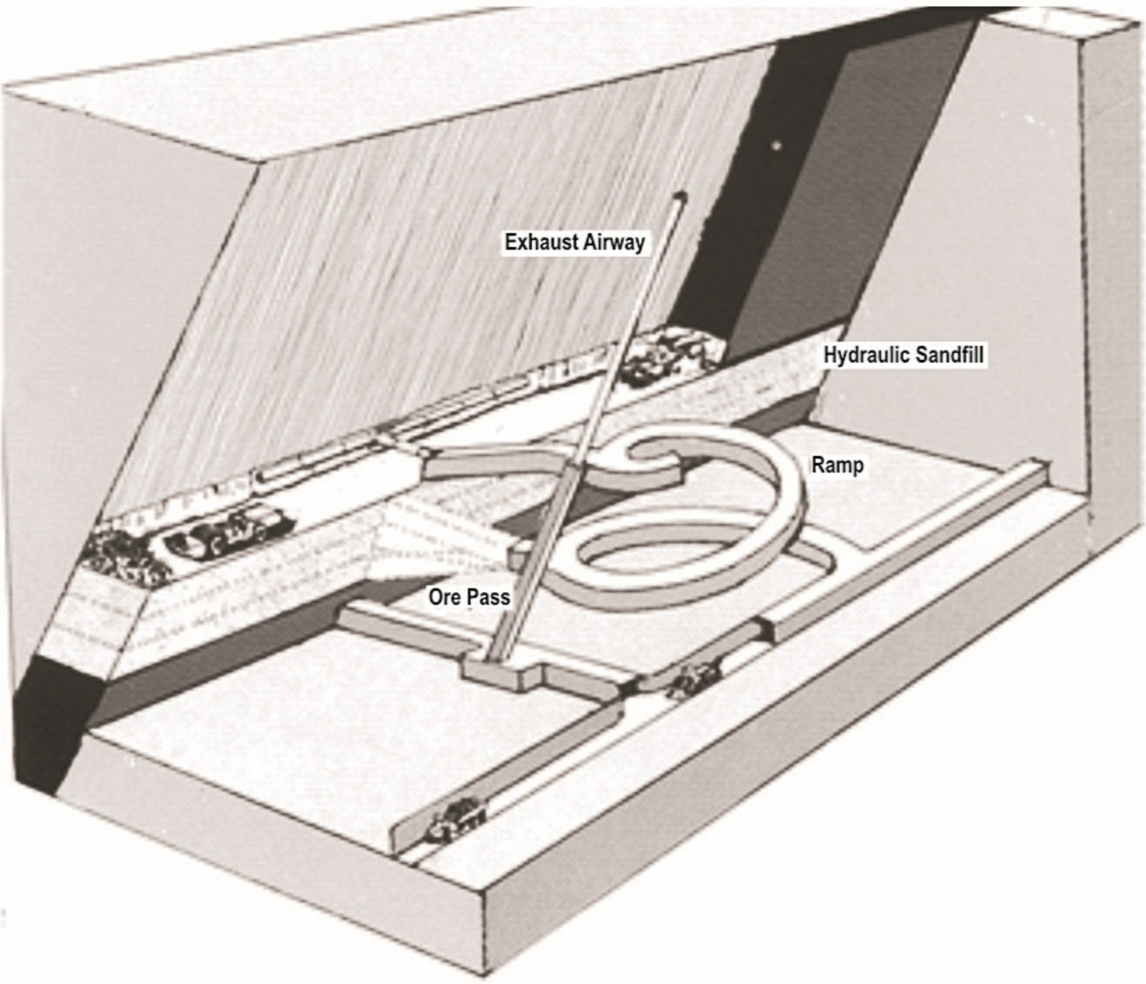


Figure 7.4 - Cut and Fill Stoping

or to make a safe roof for underhand stoping. Hydraulic placement of the fill is the most common practice, with the material prepared in a surface plant and transported underground through pipelines.

7.5 OPEN STOPING

Open stoping is a mining method for extracting small, erratic ore deposits encountered as veins, sills or mantos in relatively competent rock. The method does not require a large investment in expensive mining equipment and can be used to effectively follow ore shoots on a blast round to blast round basis. In flat lying ore zones, the footwall of the deposit is used as a working floor, while in steeply dipping deposits timber staging is commonly used to work from or pillars of low-grade material may be advantageously used as a working base.

Transport of the broken rock down to the haulage level may be by gravity alone, or by utilising slushers where the dip of the ore zones is too flat to permit 100 percent gravity movement of the blasted ore. Open stoping of small deposits permits high selectivity of the material to be mined, but daily production is generally very limited at mines employing it as the principal method of extraction. It is commonly used as a scavenging method for recovering ore that might otherwise be lost at larger mines where the principal production methods are based on blasthole or shrinkage stoping systems. It is a mining system commonly employed at smaller precious metal mines where ground conditions are good, and the orebody consists of small ore shoots. It is especially prevalent in many lesser developed nations where mining regulations are less stringent, labour costs lower and mining equipment is relatively high priced.

7.5.1 Sublevel Caving

Sublevel caving can be applied to those large ore deposits in which the ore itself is relatively strong, but the host rock is weak enough to cave when the ore is removed. The geometry of the ore deposit influences the selection of this stoping method. A steeply dipping deposit is more satisfactory for its application than a relatively flat deposit, unless the latter has considerable thickness. Sublevel caving can often be used to extract ore bodies whose limited size or rock competency precludes extraction by the block caving system, and is flexible enough to be applied to irregular ore bodies of varying widths.

The principal disadvantages of sublevel caving is the resulting high dilution of the ore caused by caving of waste material from the walls and the relatively high development cost to bring the mine into production. Since sublevel caving induces failure of the wall rock and overburden, surface subsidence results in locating all permanent structures outside of the area of influence.

Sublevel caving mines lend themselves to mechanisation and mining activities can be specialised with training imparted to underground personnel. Mining activities on each level are similar, i.e., development of the levels, production drilling on the intermediate levels and production blasting with ore extraction on the upper levels. Consequently, the supervision of the activities also gets simplified as interference between the activities gets minimised (refer Figure 7.5—Plate 10).

7.5.2 Room and Pillar Stoping

Tabular, flat dipping ore deposits in competent rock are usually mined by room and pillar stoping methods. If the ore zone is continuous over long distances, a regular pattern of support pillars can be laid out to yield maximum recovery of the ore and at the same time provide sufficient support for the hanging wall or roof. If the ore zones are erratic, random support pillars can be left in areas of waste or low-grade material. The principal advantage of room and pillar stoping is that it is readily adaptable to mechanised mining equipment, which results in high productivity at relatively low cost per ton of material extracted. For large ore bodies, a large number of working places can be easily developed so that high daily rates of production can be counted upon. Most of the mine development work is in ore so waste extraction is kept to a minimum. The main disadvantage of room and pillar mining is that a large area of roof is continuously exposed where work activities or movement of men and supplies are carried out. Consequently, roof soundness is a primary concern for the safety of personnel and ground support is generally a major concern, especially in rooms with high backs. Also, recirculation of ventilating air can be difficult to minimise in room and pillar mines (refer Figure 7.6—Plate 10).

7.6 MISCELLANEOUS STOPING METHODS

Three stoping systems that were commonly used in the past, but because of their labour intensive characteristics are no longer favoured are the *square set*, *top slicing* and *resuing methods*. The square set and top slicing methods are used in extremely poor ground where other extraction methods are not practical. Both methods require large amounts of timber and an experienced work force to be successfully implemented. Because of the large amount of timber used, both methods present a definite fire hazard for the entire mine. The characteristics of top slicing and square setting preclude mechanisation of their operations, so their application is limited to very high-grade ore bodies. Resuing is a method of stoping in which the ore is broken and removed first followed by the blasting of the waste or vice versa. Usually the material which breaks easier is blasted first. The broken waste is left in the stope as filling and a plank floor laid on the fill to prevent mixing of ore and waste. Resuing is applicable where the ore is not frozen to the stope walls and works best if there is a considerable difference between the hardness of the ore and the wall rocks. The method is labour intensive and is rarely practiced anymore, except in very high-grade, narrow vein, gold and silver deposits.

7.6.1 In situ and Induced Stresses

Rock at depth is subjected to stresses resulting from the weight of the overlying strata and from locked-in stresses of tectonic origin. When an opening is excavated in this rock, the stress field is locally disrupted and a new set of stresses are induced in the rock surrounding the opening. Knowledge of the magnitudes and directions of these in situ and induced stresses is an essential component of underground excavation design since, in many cases, when the strength of the rock is exceeded, the resulting instability have serious consequences on the behaviour of the excavations. This Chapter deals with the question of in situ stresses and also with the stress changes that are induced when tunnels or caverns are excavated in stressed rock. Problems, associated with failure of the rock around underground openings and with the design of support for these openings, will be dealt with in later chapters. The presentation, which follows, is intended to cover only those topics which are essential for the reader to know about when dealing with the analysis of stress induced instability and the design of support to stabilise the rock under these conditions.

In situ Stresses

Consider an element of rock at a depth of 1,000 m below the surface. The weight of the vertical column of rock resting on this element is the product of the depth and the unit weight of the overlying rock mass (typically about 2.7 tonnes/m^3 or 0.027 MN/m^3). Hence the vertical stress on the element is 2,700 tonnes/m^2 or 27 MPa. This stress is estimated from the simple relationship:

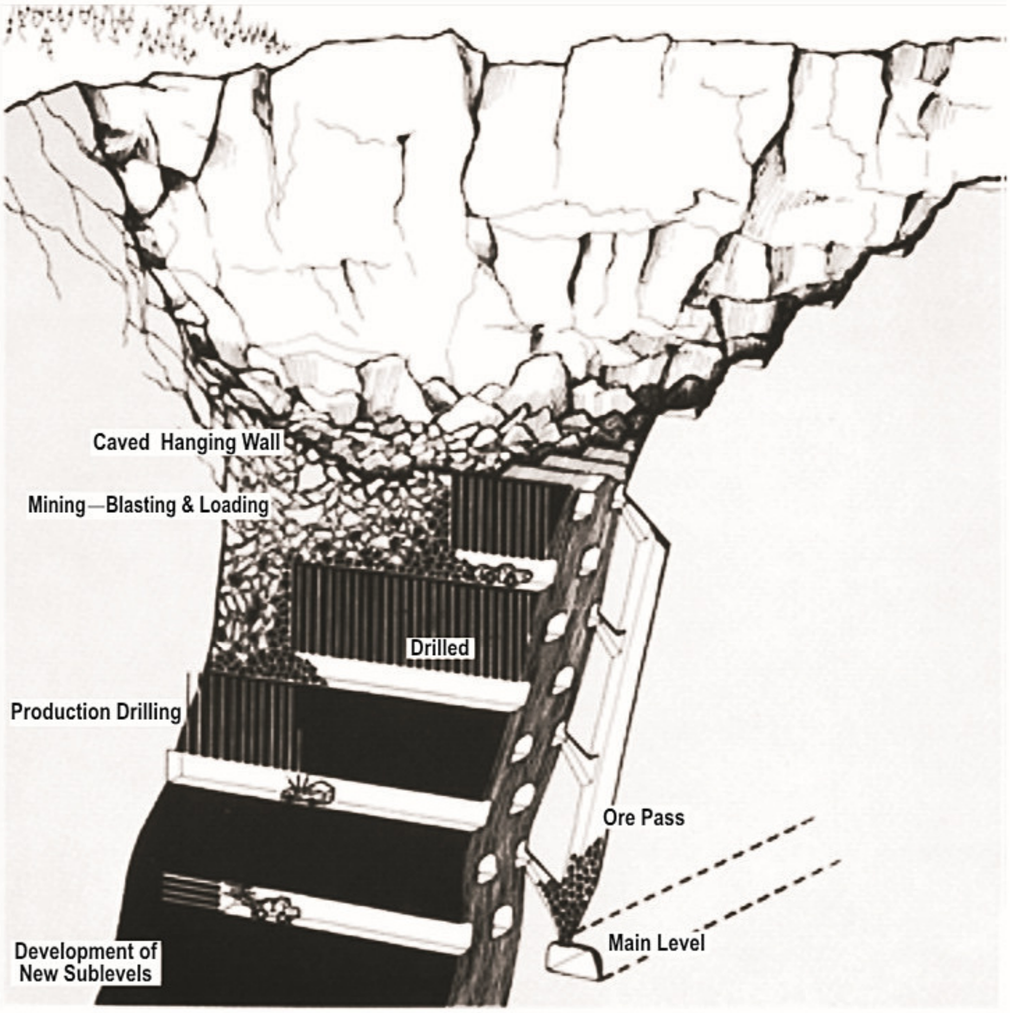


Figure 7.5 - Sublevel Caving

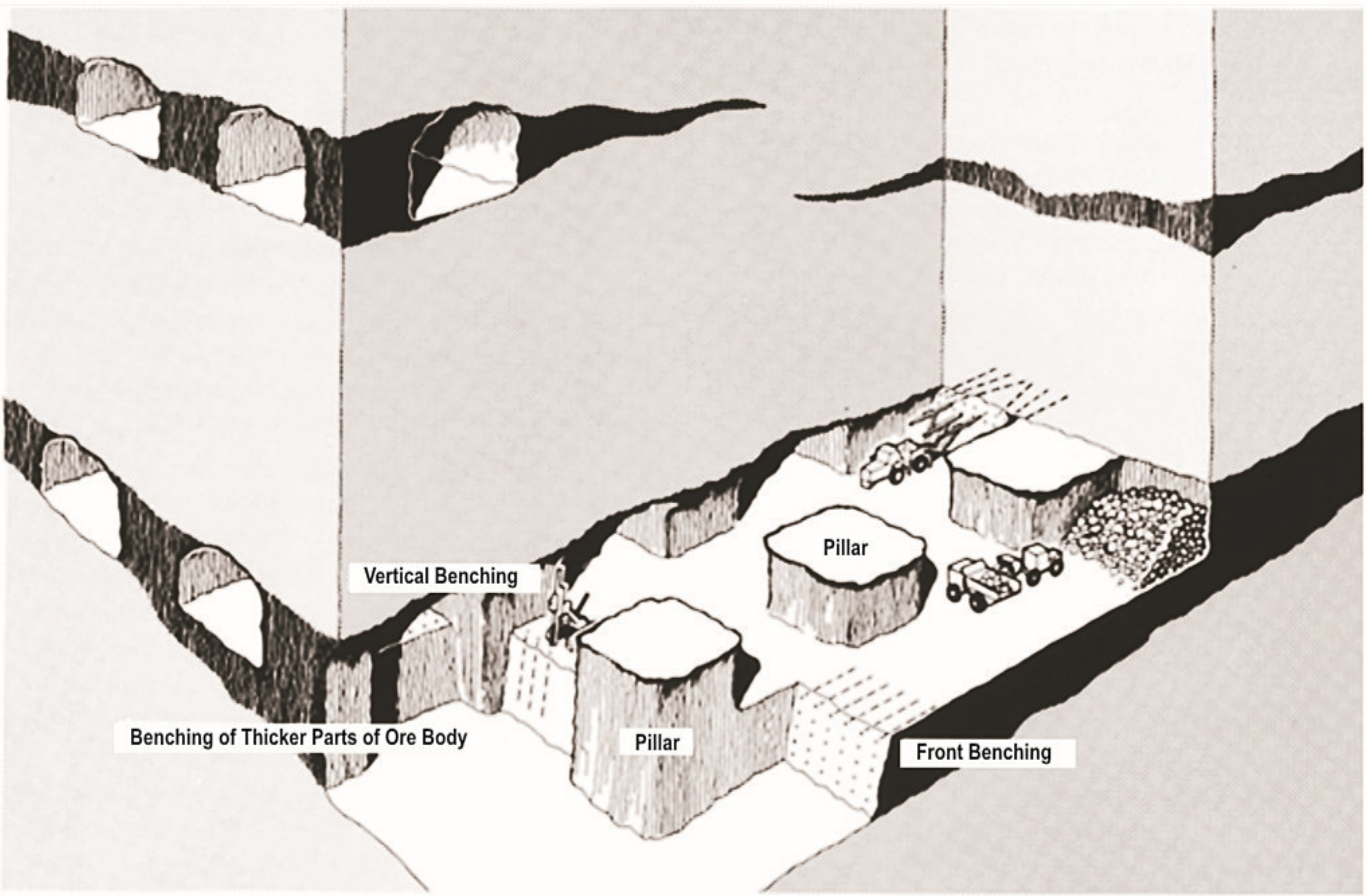


Figure 7.6 – Room and Pillar Stoping

$$\sigma_v = \gamma z \qquad (1)$$

where, σ_v is the vertical stress,

γ is the unit weight of the overlying rock, and

z is the depth below surface.

Measurements of vertical stress at various mining and civil engineering sites around the world confirm that this relationship is valid although, as illustrated in Figure 7.7, there is a significant amount of scatter in the measurements.

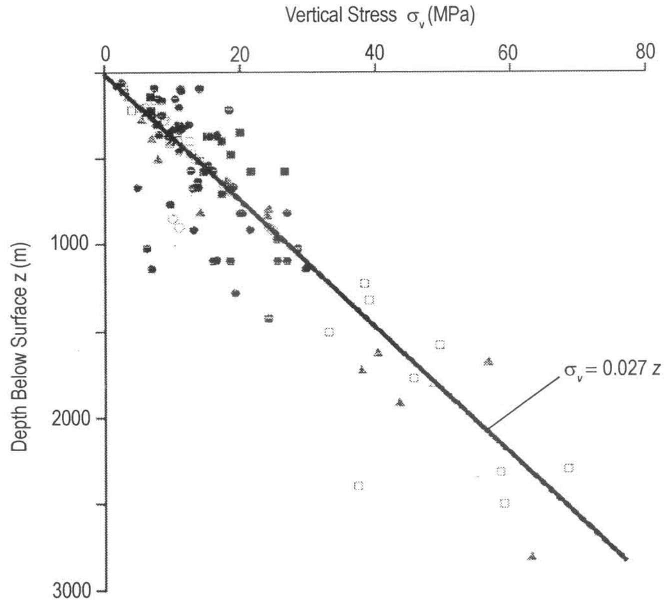


Figure 7.7 : Vertical Stress Measurements from Mining and Civil Engineering Projects Around the World (After Brown and Hoek 1978)

The horizontal stresses acting on an element of rock at a depth z below the surface are much more difficult to estimate than the vertical stresses. Normally, the ratio of the average horizontal stress to the vertical stress is denoted by the letter k such that:

$$\sigma_h = K\sigma_v = K\gamma z \qquad (2)$$

Terzaghi and Richart (1952) suggested that, for a gravitationally loaded rock mass in which no lateral strain was permitted during formation of the overlying strata, the value of k is independent of depth and is given by k= n (1- n), where n is the Poisson's ratio of the rock mass. This relationship was widely used in the early days of rock mechanics but, as discussed below, it proved to be inaccurate and is seldom used today.

Measurements of horizontal stresses at civil and mining sites around the world show that the ratio k tends to be high at shallow depth and that it decreases at depth (Brown and Hoek, 1978, Herget, 1988). In order to understand the reason for these horizontal stress variations it is necessary to consider the problem on a much larger scale than that of a single site.

Sheorey (1994) developed an elasto-static thermal stress model of the earth. This model considers curvature of the crust and variation of elastic constants, density and thermal expansion coefficients through the crust and mantle. A detailed discussion on Sheorey's model is beyond the scope of this chapter, but he did provide a simplified equation which can be used for estimating the horizontal to vertical stress ratio k. This equation is:

$$k = 0.25 + 7 E_h \left(0.001 + \frac{1}{z}\right) \tag{3}$$

where z (m) is the depth below surface and E_h (GPa) is the average deformation modulus of the upper part of the earth's crust measured in a horizontal direction. This direction of measurement is important particularly in layered sedimentary rocks, in which the deformation modulus may be significantly different in different directions. A plot of this equation is given in Figure 7.8 for a range of deformation moduli. The curves relating k with depth below surface z are similar to those published by Brown and Hoek (1978), Herget (1988) and others for measured in situ stresses. Hence, Equation 3 is considered to provide a reasonable basis for estimating the value of k.

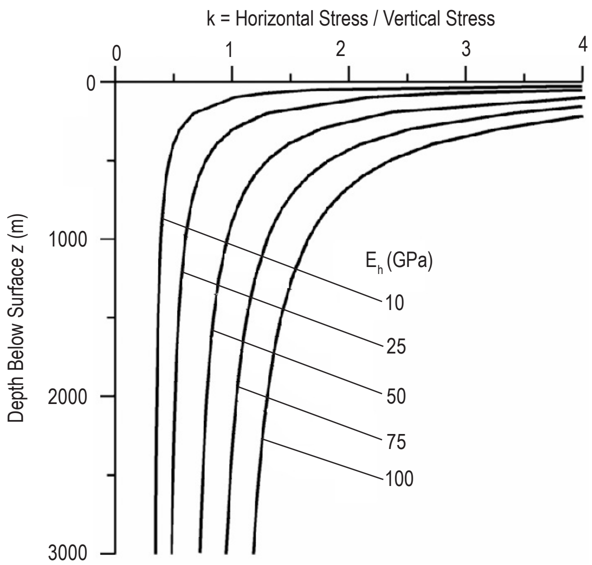


Figure 7.8: *Ratio of Horizontal to Vertical Stress for Different Deformation Moduli Based Upon Sheorey's Equation (After Sheorey 1994)*

As pointed out by Sheorey, his work does not explain the occurrence of measured vertical stresses that are higher than the calculated overburden pressure, the presence of very high horizontal stresses at some locations or why the two horizontal stresses are seldom equal. These differences are probably due to local topographic and geological features that cannot be taken into account in a large scale model such as that proposed by Sheorey.

Where sensitivity studies have shown that the in situ stresses are likely to have a significant influence on the behaviour of underground openings, it is recommended that the in situ stresses should be measured. Suggestions for setting up a stress measuring programme are discussed later in this chapter.

1. Flatjack

$$\begin{bmatrix} \sigma_{xx} & \tau_{xy} & \tau_{xz} \\ & \sigma_{yy} & \tau_{yz} \\ \text{Symm.} & & \sigma_{zz} \end{bmatrix}$$

One normal stress component determined, say parallel to x-axis.

2. Hydraulic Fracturing

$$\begin{bmatrix} \sigma_1 & 0 & 0 \\ & \sigma_2 & 0 \\ \text{Symm.} & & \sigma_3 \end{bmatrix}$$

Principal stresses assumed parallel to axes i.e. plane of the fracture, two determined, say σ_1 and σ_3, one estimated, say σ_2

3. USBM Overcoring Torpedo

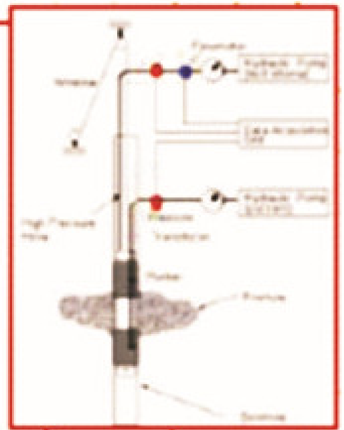
$$\begin{bmatrix} \sigma_{xx} & \tau_{xy} & \tau_{xz} \\ & \sigma_{yy} & \tau_{yz} \\ \text{Symm.} & & \sigma_{zz} \end{bmatrix}$$

Three components in 2-D determined from three measurements of borehole diameter change.

4. CSIRO Overcroring Gauge

$$\begin{bmatrix} \sigma_{xx} & \tau_{xy} & \tau_{xz} \\ & \sigma_{yy} & \tau_{yz} \\ \text{Symm.} & & \sigma_{zz} \end{bmatrix}$$

All six components determined from six (or more) measurements of strain at one time.



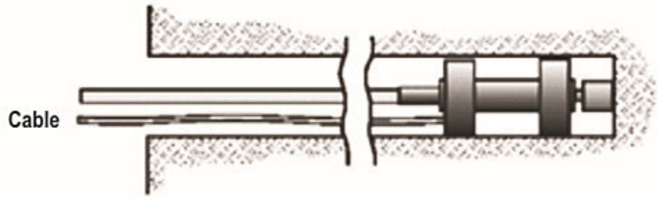
Hudson & Harrison (1997)

The four ISRM suggested methods for rock stress determination and their ability to

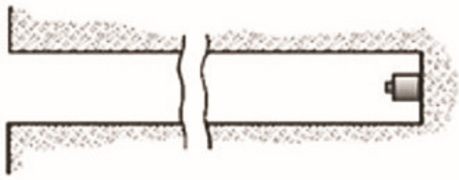
Dr. Erik Eberhardt

Figure 7.9 – Methods of In situ Stress Determination in Underground Mines

A diamond drill hole (76 mm outer diameter) is drilled to wanted depth. The core is removed and the hole bottom is flattened with a special drill bit.



A two dimensional measuring cell (doorstopper) that contains a strain gauge rosette, is inserted into the hole with a special installing tool and glued to the bottom of the hole.

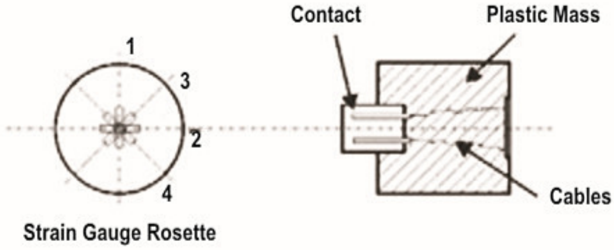


The doorstopper is now fixed to the hole and initial reading (0 recording) is done. The installing tool is removed and the cell is ready for overcoring.

A new core is drilled with the 76 mm Ø diamond drill, thus stress relieving the bottom of the borehole. The corresponding strains at the end of the core are recorded by the strain gauge rosette.

The core is caught with a special core catcher, and immediately after removal from the hole the second recording is done. From the recorded strains the stresses in the plane normal to the borehole, may be calculated when the elastic parameters determined from laboratory tests are known.

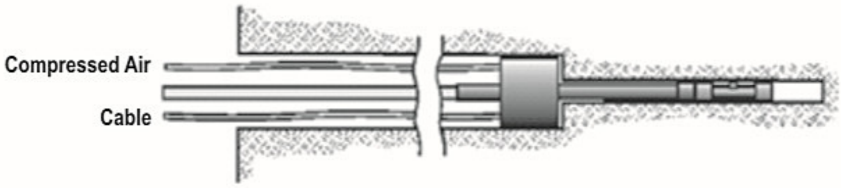
Measuring Cell "Doorstopper"



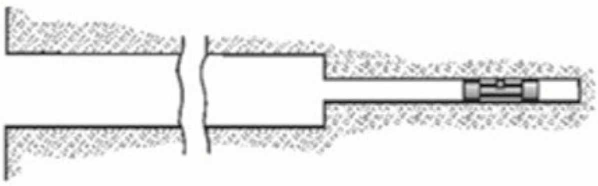
Cross and Longitudinal Section

Figure 7.10 – 2-Dimenational Rock Stress Measurements by Overcoring

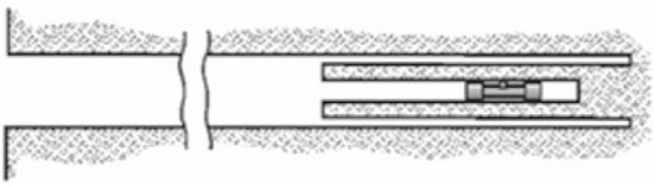
A diamond drill hole (77 mm outer diameter) is drilled to wanted depth. The hole bottom is flattened with a special drill bit, and a concentric hole with smaller diameter (36 mm o.d.) is drilled approximately 30 cm further.



The measuring cell is inserted with a special installing tool containing an orienting device and a cable to read-out unit. The instrument head is put in place with detachable aluminium rods. Compressed air is used to expand the cell in the hole, and the strain gauges are cemented to the hole wall.



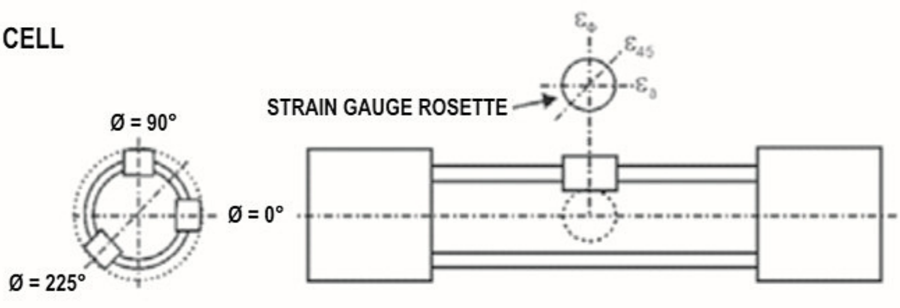
The measuring cell is now fixed to the hole and initial reading (0 recording) is done. The installing tool is removed and the cell is ready for overcoring.



The small hole is overcored by the larger diameter bit, thus stress relieving the core. The corresponding strains are recorded by the strain gauge rosettes.

The core is caught with a special core catcher, and immediately after removal from the hole, the second recording is done. From the recorded strains, the stresses may be calculated when the elastic parameters determined from biaxial - and laboratory tests are known.

MEASURING CELL



MEASURING CELL CROSS SECTION

MEASURING CELL LONGITUDINAL SECTION

Figure 7.11 - 3-Dimentional In situ Rock Stress Measurements by Overcoring

7.6.2 In situ Rock Testing Methods

There are several methods for measuring the in situ stress in underground mines. The details of various methods deployed are given in Figure 7.9 (See Plate 11).

Specimen testing at the laboratory scale does not represent properties of in situ rock due to geological factors, such as, bedding, factures, joints, faults, folding and microscopic and megascopic in homogenities. This problem can be solved by using in situ measurements to know properties of in situ rocks. To solve the problem of stability of the structure in rocks, the accuracy of stress and strain determination must be consistent with in situ rock. Sample collected from uniform and massive rocks, compressive and tensile strength measured at the laboratory scale should not vary much from specimen to specimen. In mining, adjacent area always affects rock strength. Therefore, changes in stresses have to be measured with high accuracy.

The rock stress measurements are quite often only part of larger projects, and SINTEF is one such research organisation that offers and provides rock deformation measurements (Borehole extensometer and tunnel convergence measurements), rock bolt testing, numerical modelling, and civil and mining engineering solutions in general. A number of state-of-the-art 2D and 3D numerical modelling codes are available. The 2-dimensional Rock Stress Measurements by Overcoring method are shown in Figure 7.10 (See Plate 12) & 3-D In situ Rock Stress Measurements by Overcoring method are shown in Figure 7.11 (See Plate 13).

7.6.3 Hydraulic Fracturing for Rock Stress Determination

The hydraulic fracturing technique is used for determining the in situ rock stress in a plane perpendicular to borehole. This is done by application of fluid pressure (normally water) in a test section in a borehole isolated by packers until the rock fails in tension. The fluid pressures required to generate, propagate, sustain and reopen tensile fractures in the rock are recorded as function of time, and these may be related to magnitude of the existing stress field. Directions of measured stress are normally achieved by observing and measuring the orientation of the hydraulically induced fracture plane by the use of a so-called impression packer. The induced hydro fracture is oriented parallel with the major secondary principal stress σ_H in a plane perpendicular to the borehole. The time/fluid pressure recording data is presented in Figure 7.12 & hydraulic fracturing equipment is shown in Figure 7.13 (See Plate 14).

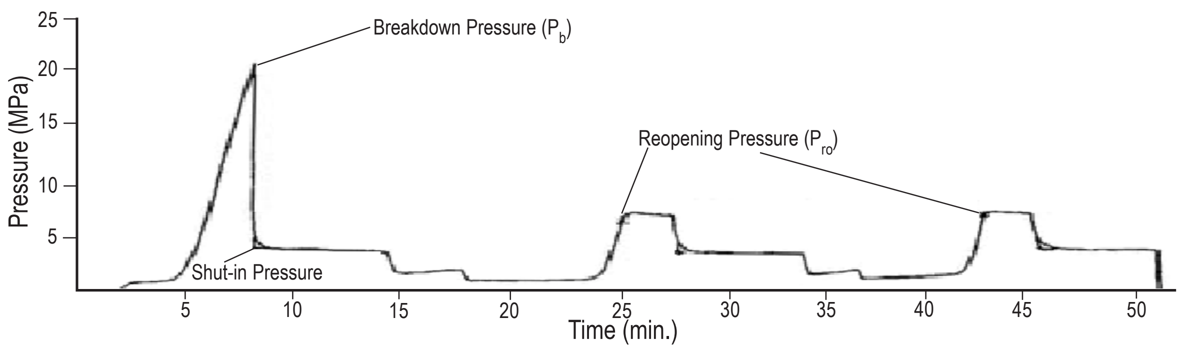


Figure 7.12: *Time/Fluid Pressure Recording*

Hydraulic fracturing is usually carried out in vertical holes drilled from the surface or underground in holes drilled from tunnels (vertical, inclined or horizontal). It could be combined with overcoring from tunnels, i.e. hydraulic fracturing could be carried out after overcoring in the same hole.

- Maximum borehole length: 250 m
- Borehole diameter: 46–76 mm
- Computerised logging equipment may be used in combination with overcoring in holes drilled from tunnels
- Equipment units may be shipped by normal air freight

7.6.4 Determination of Absolute Stress

The flatjack method involves the placement of two pins fixed into the wall of an excavation. The distance, d, is then measured accurately. A slot is cut into the rock between the pins. If the normal stress is compressive, the pins will move together as the slot is cut. The flatjack is then placed and grouted into the slot. The borehole deformation method, flatjack method, propagation method have been employed by several investigators to determine the absolute stress in rock (refer Figure 7.14—Plate 14).

a) A small diameter hole approximately 1" is drilled in to an underground rock surface. The borehole deformation gauge is then inserted into the hole, radially oriented and an initial reading is recorded.

b) The section of core containing the gauge is concentrically overcored with a larger diameter bit, thereby relieving the stress from the core-containing gauge. A second gauge reading recorded.

c) The difference between first and second gauge reading is the borehole deformation.

d) The gauge is moved to a point further in the gauge hole, oriented at 60° from the original position and overcoring procedure is repeated. Thus, the procedure is cyclically continued for the 60° rosette position till the desired range of depth has been investigated. The magnitude and direction of the maximum and minimum stress in the plain normal to the axis of the gauge hole can be calculated as follows:

$$P = (E/6d) \times \{(U_1+U_2+U_3)+(2^{0.5}/2)[(U_1-U_2)^2+(U_2-U_3)^2+(U_3-U_1)^2\ ]^{0.5}\}$$

$$Q = (E/6d) \times \{(U_1+U_2+U_3)-(2^{0.5}/2)[(U_1-U_2)^2+(U_2-U_3)^2+(U_3-U_1)^2\ ]^{0.5}\}$$

$$Q_{pq} = 0.5 \tan^{-1} [3(U_2-U_3)/(2U_1-U_2-U_3)]$$

where, P = Maximum Principal Stress

Q = Minimum Principal Stress

Q_{pq} = Angle measured from U_1 to P in counter clockwise direction or magnitude of applied stress

d = diameter of borehole

U_1, U_2, U_3 = Set of three diametral measurements.

E = Modulus of elasticity of the rock

In some modified procedure, surface mounted strain gauges are replaced with copper foil jacketed resistance-strain gauges grouted in the slot cut above and below the intended flat jack slot. These gauges are placed directly over and under the centre of the flat jack and oriented so that they will measure strain in the direction normal to the flat jack. In some modified procedure, resistance strain gauges are replaced by the hydraulic cells.

Flatjack method does not require the elastic properties of rock. Hence it is considered the true stress measuring method. Due to difficulty in cutting deep flatjack slot, the method is restricted to near surface measurements. The method is also used in inelastic rock.

7.6.5 Determination of In situ Stresses by Flatjack Method

The method is intended for the determination of rock stress parallel to and near the exposed rock surface in an excavation. Each measurement determines stress in one direction only and therefore a minimum of six measurements in independent directions are required to determine the rock stress tensor.

The method involves the observation of the movements of pairs of measuring pins located on each side of a slot when the slot is cut and subsequently when pressure is applied to the internal surface of the slot.

Typical application of flatjack tests include:

In tunnels	:	actual loads on tunnel linings
In situ stress	:	determination of excavation induced stress
In buildings	:	actual loads on pillars and footings
In dam abutments	:	In situ rock stresses often carried out as part of large flatjack testing

The stress determination with flatjacks results in determination of the disturbed stress components in the immediate vicinity of the opening. This information can be extrapolated from the opening outward to the undisturbed virgin stress by application of the theory of elasticity or by numerical modelling techniques.

The slot closure and opening values are calculated for each pair of pins and for each pressurisation increment by subtracting initial from subsequent readings. The closure and opening for each pair of pins are plotted against applied pressure to determine the average cancellation pressure (see Figure 7.15).

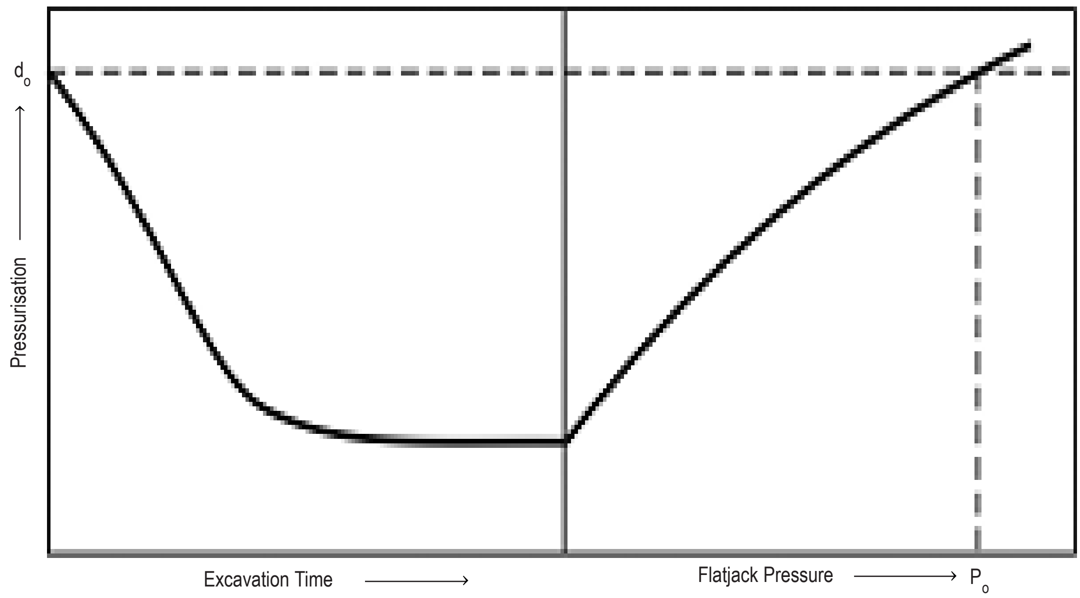


***Figure 7.15**: Determination of Average Cancellation Pressure*

This cancellation pressure gives the normal stress across the slot. The orientation of the slot and magnitude of normal stress across each slot are measured. Readings from six such tests located in independent directions are taken for the determination of stress tensor at the particular site.

Limitations

a) The stress relief is assumed to be an elastic, reversible process. In non-homogeneous or highly fractured materials, this may not be completely true.

b) The flatjack is assumed to be 100% efficient.

c) The jack is assumed to be aligned with the principal stresses on the surface of the opening. Shear stresses are not cancelled by jack pressure.

d) The flatjack test measures stresses only at the surface of the test chamber. Undisturbed stress levels must be determined by theoretical interpretations of these data.

Site Preparation

At the selected site all the slacked rocks are removed and the surface of tight rock is rimmed to nearly flat and horizontal. Tight rock surfaces are washed and cleaned. The locations of the slots are marked on the wall and the pairs of measuring pins are installed across the marks by drilling holes by a masonry drill machine and grouted. LVDTs are installed across the proposed slots by fixing the body and the piston of the LVDT against the grouted pins by appropriate clamps. Specifications for site preparation are shown in Figure 7.16 (See Plate 14).

7.7 EQUIPMENT

Flatjack	LVDTs	Measurement Pins-groutable Type	Hydraulic Pump
a) Width: 350 mm b) Length= 400 mm c) Thickness= 6 mm (deflated) d) Pressure range = 0–20 MPa	a) Diameter = 20 mm b) Reading Accuracy = 0.002 mm c) Voltage Output = 0–10 V	12 mm diameter and 150 mm length with holders for holding the potentiometers	Pressure range: 0–70 MPa

7.7.1 Test Procedures

After setting of the LVDTs across the marked slot position, readings are taken for initial separation of the pins. The slot is then cut by drilling a series of 42 mm holes along the slot up to a depth slightly more than the flatjack width. Direction of the slot is then measured using a magnetic compass. Further sets of displacement readings are taken after cutting the slot to record the amount of slot closure. The flatjack is then inserted fully and grouted.

After the grout is set, a hydraulic pump increases the pressure in the flat jack. Pressure is increased gradually and reading for each increment of pressure is noted. Pressure is increased until the separation of pins is the same as before the slot was cut. The pressure at which it is achieved is termed as cancellation pressure. After achieving the cancellation pressure the whole set up is then moved to the next slot position and the whole procedure is repeated. It is recommended to carry out six numbers of flatjack test at a site to know the state of stress at the site. Flatjack installed at one of the test sites is shown in Figures 7.17 & 7.18 (See Plate 15).

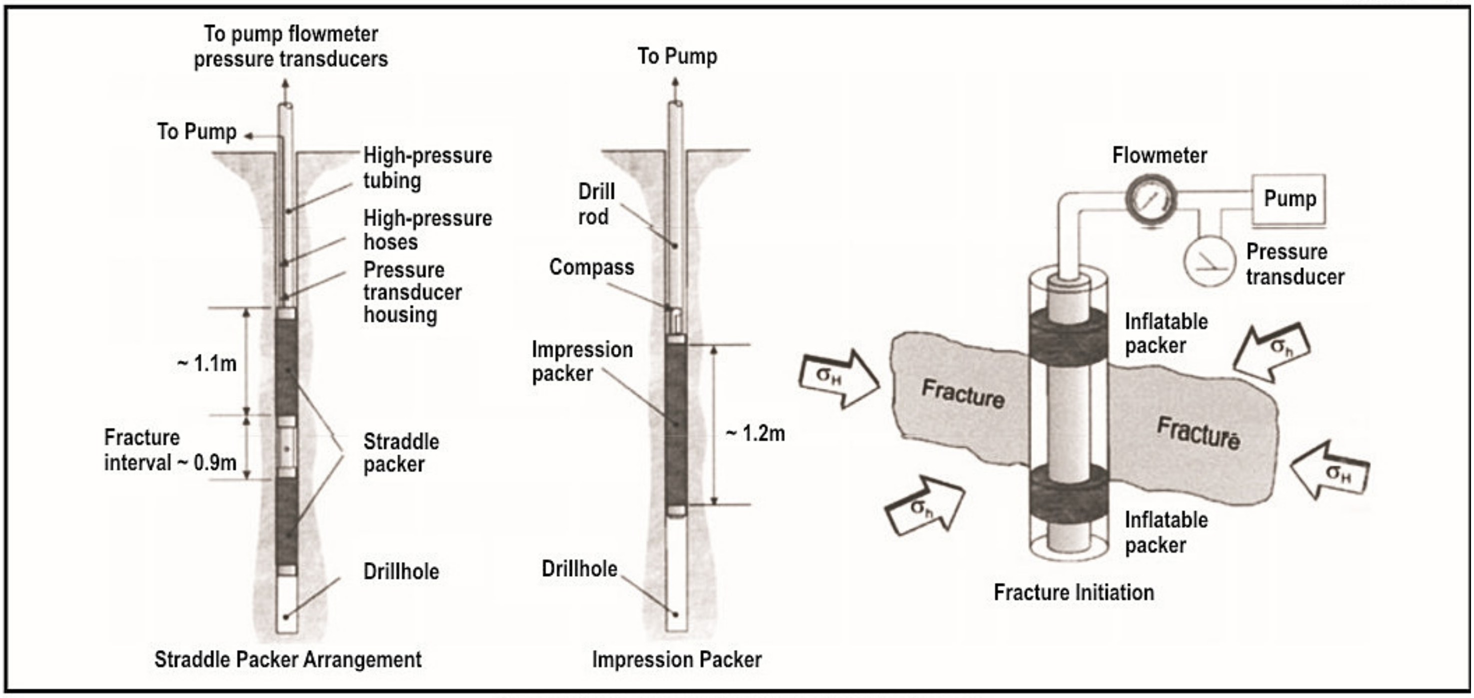


Figure 7.12 – Hydraulic Fracturing Equipment

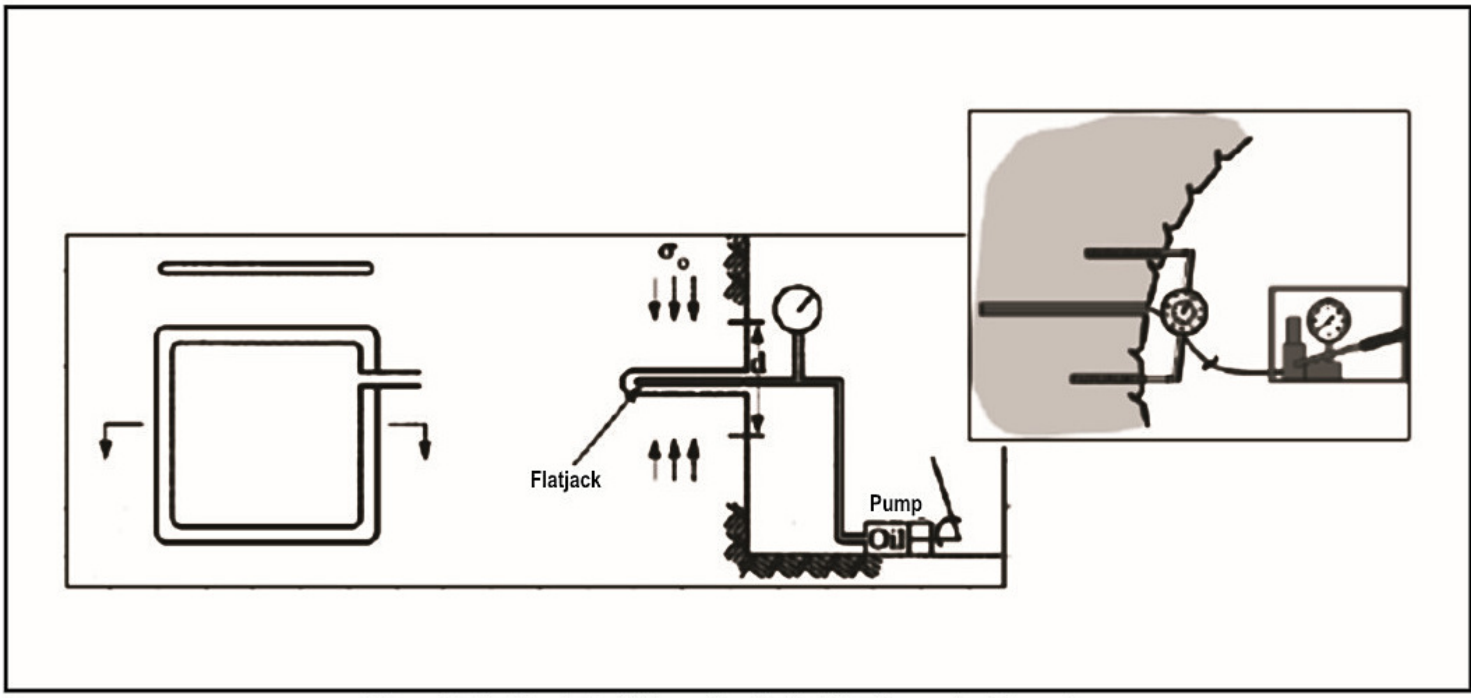


Figure 7.14 - Borehole Deformation Method by Overcoring Procedure.

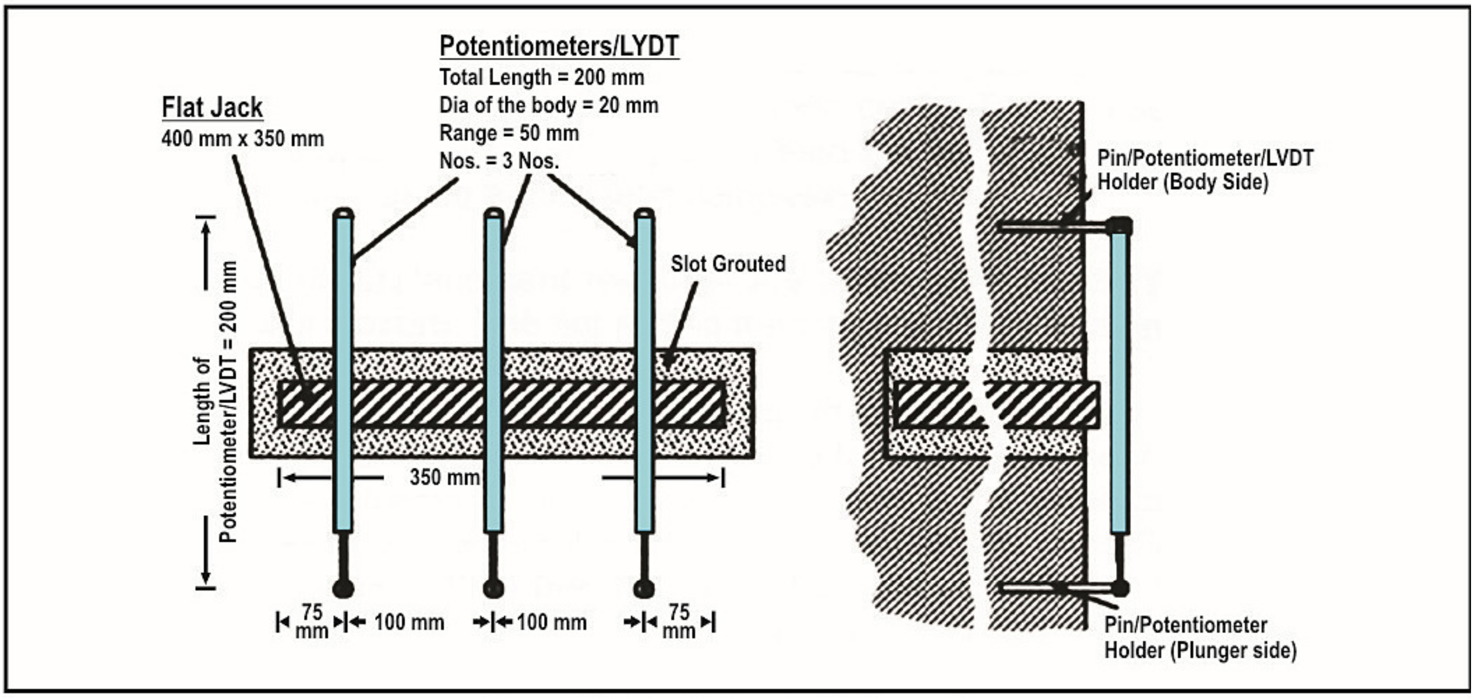


Figure 7.16 - Specifications for Site Preparation

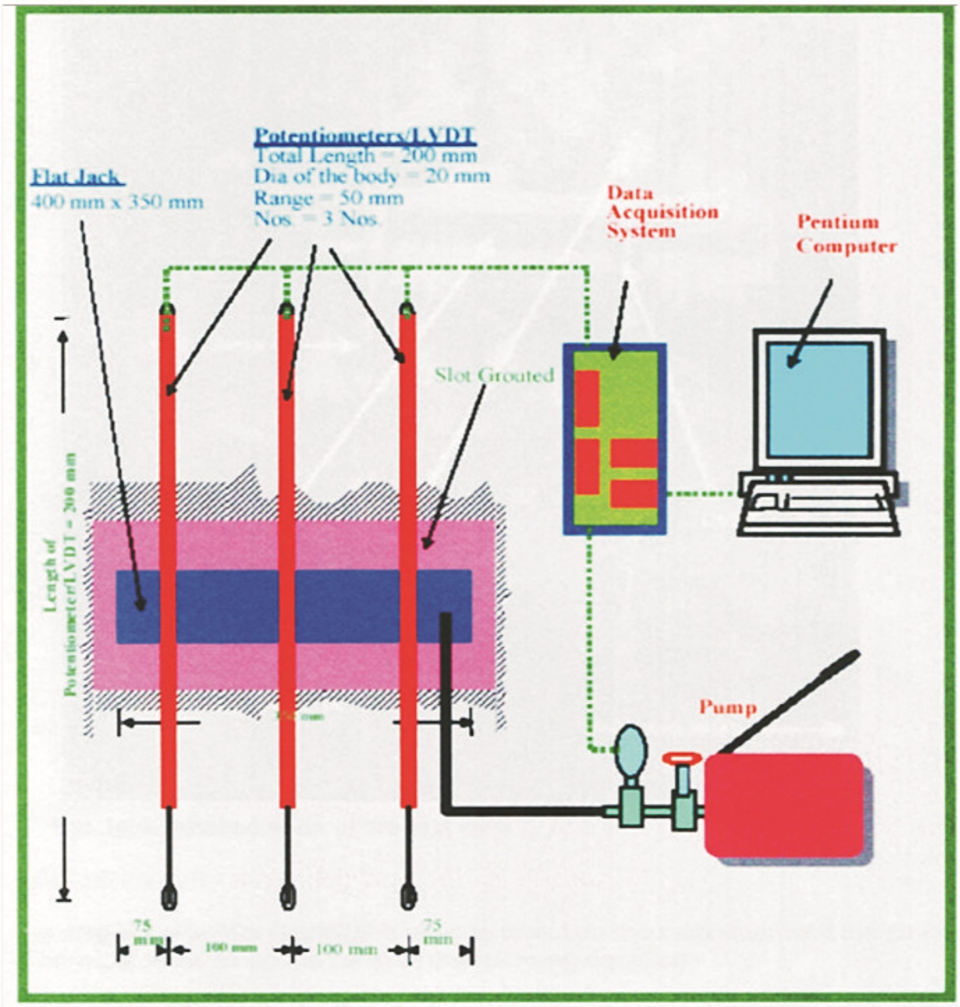


Figure 7.17 - Schematic Diagram for Flat Jack Test

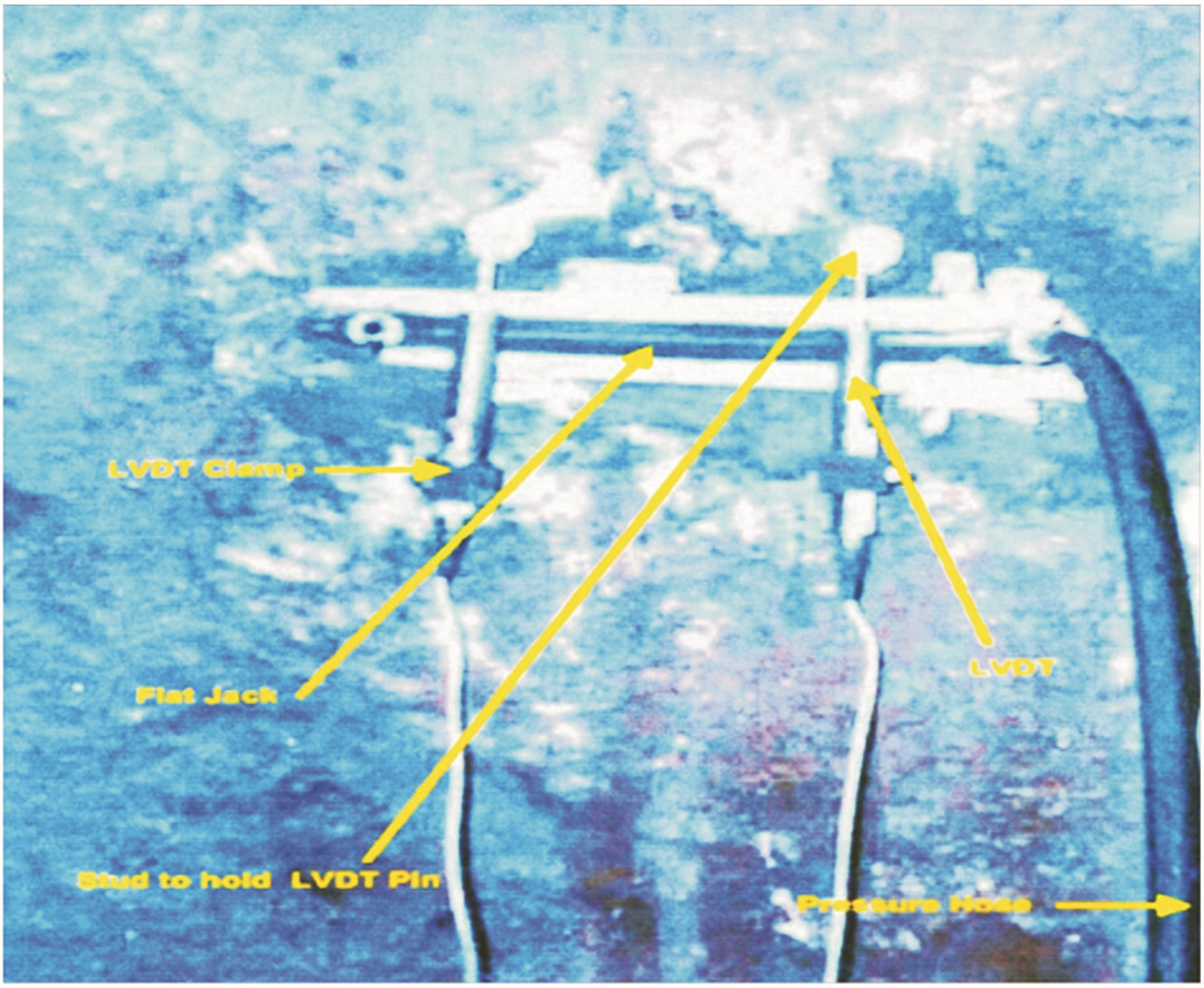


Figure 7.18 - Flat Jack Installed at on of the Test Sites.

7.7.2 Stress Evaluation Procedure and Results

The analysis programme GENSIM is used to calculate the magnitude and the direction of principal stresses on the basis of the following equation:

$$\sigma h = (P - n^2 . \sigma V) / (m^2 + l^2 . \sigma H/\sigma h)$$

where, *l, m* & *n* are the cosines of the direction of the induced fracture plane related to the principal stress axis. The calculations of stress for each site are done by using all cancellation pressure data and varying the ratio σH/σh and the strike direction of σH in the horizontal plane.

7.8 DETERMINATION OF IN SITU DEFORMABILITY OF ROCK BY GOODMAN JACK TEST

The Goodman Jack is a borehole probe with moveable rigid bearing plates for the measurement of wall deformation as a function of applied stress. Data obtained from the load-deformation measurements gives the moduli of rock directly. The probe is designed to be used in a NX (76 mm) borehole. Hydraulic pressure is transmitted to the rock through the moveable plates. Two LVDT displacement transducers are mounted within the jack at each end of the moveable plates.

7.8.1 Application of Goodman Jack Test

Goodman Jack tests find the following applications:

a) These tests are required during the design of large underground excavations since such tests provide the rock mass deformability.

b) Estimation of the grouting efficiency

c) Estimation of rock anisotropy.

d) Estimation of rock behaviour during frequent loading and unloading process.

The result of Goodman Jack testing in a borehole of diameter 'd' is a graphical plot of applied pressure 'Q_H' versus diametric deformation 'ud'. Young's modulus will be computed as the tangent modulus along the slope of linear portion of this plot of the field data.

$$E = 0.86\, K(v)\, \{\Delta Q/(\Delta ud /d)\}$$

where, ΔQ = incremental change in plate pressure (MPa)

Δud = incremental change in displacement, in mm

d = borehole diameter in mm

K(v) = jack constant, depending upon Poisson's ratio for the material being tested.

For a jack efficiency of 93%, the applied hydraulic pressure, Q_H, is greater than the plate pressure. For the Goodman Jack (Model 52101)

$$Q = 0.93\, Q_H$$

K(v) is a factor dependent on the Poisson's ratio for the rock being tested. But the term *T* is adopted instead of *K(v)* for the full contact of the jack and are tabulated in the following table (Ref. Goodman. Van and Heuze, 1968).

T* for full contact

Poisson's ratio (v)	0.1	0.2	0.25	0.3	0.33	0.4	0.5
T*	1.519	1.474	1.438	1.397	1.366	1.289	1.151

Also, the formula in the above table should be altered to be an expression for the calculated apparent modulus of deformation, E calc as follows:

E calc = 0.80 (d)∗{ ΔQ H/ Δud} ∗ T*

After calculating Ecalc the same is converted in E true from the plot given in Figure 7.19. This curve is not sensitive to variations in v. For E calc less than 7 GPa the correction from E calc to E true is negligible and E calc can be taken as equal to E true.

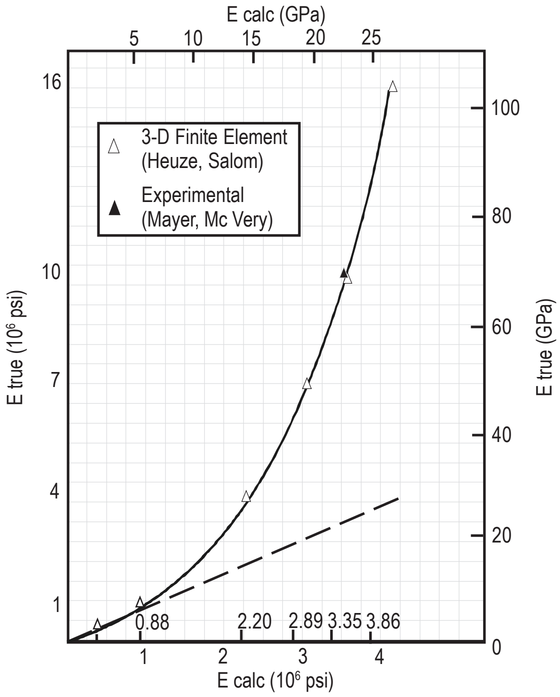


***Figure 7.19**: Curve of E calc versus E true*

7.8.2 Site Preparations

The jack is designed to be used in NX size boreholes which have a nominal diameter of 76.2 mm. The borehole must be core drilled so that proper zones can be selected by inspecting the cores.

Specifications of Goodman Jack

Effective Jack Force: The area of 'T' the operating pistons in the jack is such that the maximum hydraulic pressure of 68.95 MPa produce the following forces uni-directionally against the rock. Bearing plate pressure = 64.12 MPa and Total Force against rock = 702.8 kN.

LVDTs: The two linear variable differential transformer (LVDT) displacement transducers are mounted within the jack. These are Trans-Tek Model 241-000, which have a linear range of 2.54 mm. The linearity over this range is within ± 0.5% of the full scale linear range. The maximum usable displacement range of this LVDT is ± 0.15% (3.81 mm).

Displacement Indicator: The portable instrument, Model 52127, operates on internal, rechargeable battery or on 110 volt AC. The displacement of the pressure plate is indicated by two illuminate digital displays, one for each LVDT. These displays give the deviation of the plate from the nominal borehole diameter of 76.2 mm with a sensitivity of 0.01 mm.

Jack: The Goodman Jack consists of two movable rigid bearing-plates. The total displacement or extension of the jack is 11.4 mm. When the jack is fully closed, the diametrical distance between the outside surfaces of the two pressure plates is 69.9 mm. When the jack is fully open, the diametrical distance is 81.2 mm. The jack and the hydraulic components are designed for a 68.95 MPa maximum working pressure. The hydraulic pump (Enerpac P-84) produces a maximum output pressure of 68.95 MPa.

Pressure Gauge: The pressure gauge (Marsh Type 200) has a bourdon tube sensing element. The accuracy of this gauge is ± 0.25% of full scale. The full scale pressure is 70 MPa. The smallest division of the scale is equal to 0.5 MPa.

7.8.3 Test Procedure

The Goodman jack is inserted into the borehole to the desired test depth. The indicator power is switched to the ON position and the pump valve is set in extended position. The pressure is then applied until the jack expands against the sides of the borehole. The meter reading is observed while the jack pressure plate is being extended up to zero reading. When the contact with the side of the jack is made, the pressure is increased until the pressure gauge indicate a value equal to the first increment. Ten such increments are made. The meter reading for both the LVDTs are noted. The pressure is kept constant with the pump handle recording the meter reading every minute. This is continued until the displacement became constant.

The peak pressure is reached in three cycles (approximately 30, 60 and 100% of the maximum). During each cycle the pressure is varied in at least ten equal increments and decrement. At the end of each cycle the pressure is returned to the initial seating pressure. The tests can be conducted in different directions to determine the anisotropism of the rock.

7.9 DETERMINATION OF IN SITU PERMEABILITY OF ROCK BY DOUBLE PACKER PERMEABILITY TEST

The permeability of the rock mass is an important feature in the assessment of groundwater inflow into underground workings. Besides that the method is used for assessing the need for foundation grouting. It comprises calculations of lugeon values for each of the five test runs at increasing and then decreasing pressures followed by interpretation of the pattern of results, and hence selection of an appropriate representative permeability.

7.9.1 Application of Permeability Test

Typical applications include for the assessment of foundation to decide when grouting is warranted. Estimation of ground water flow into the underground workings.

a) Five consecutive water (pump-in) tests are done, each of ten minutes duration.

1st 10 minutes is at low pressure	-	(pressure “a”)
2nd 10 minutes run is at medium pressure	-	(pressure “b”)
3rd 10 minutes run is at a peak pressure	-	(pressure “c”)
4th 10 minutes is at a medium pressure	-	(pressure “b” again)
5th 10 minutes run is at low pressure	-	(pressure “a” again)

b) A single lugeon value is then calculated for each one of these five tests, using the formula:

Lugeon value = water taken in test (litres/meter/minute) X 10 (bars) (test pressure(bars)

c) Having calculated the five lugeon values, they are inspected and compared and an appropriate decision can then be made as to which of the five values is accepted as the permeability reported from the test.

Lugeon (1933) in his standard test, specified a pressure of 10 bars (1000 KPa). The "modified" test usually uses lower pressure than this because:

(i) a range of pressures (rather than a single pressure) is desirable.

(ii) use of a pressure as high as 10 bars is not always advisable, particularly at shallow depths in the weaker rocks.

(iii) satisfactory results can be obtained with lower pressures.

When using the "modified" test, it is necessary to convert the results into values which would have been (supposedly) obtained if the "definition" pressure (10 bars) had been used. Lugeon values then result from this conversion.

7.9.2 Site Preparation

The Double Packer System is designed to be used in NX size boreholes which have a nominal diameter of 76.2 mm. The borehole must be core drilled so that proper zones can be selected by inspecting the cores.

7.9.3 Equipment to be Deployed

The double packer permeability testing equipment is meant for fast and reliable permeability tests in boreholes. The tests are conducted in the zone between the two packers by regulating the flow of water from the surface and collecting the data by a online data acquisition system. The different components of the system are as follows:

a) Double Packer System: The packer is comprised of the inflatable gland and an inner core pipe, which runs the entire length of the packer and on which two sealing heads are mounted. The deflated and inflated diameters of the packers range from 72 mm to 160 mm.

b) Perforated Spacer Pipes: These pipes are required to transmit water from the pump to the formation whose permeability is required to be determined. The pipes are typically of 3 m each and are attached to one another to have the desired length.

c) Pressure Transducer: The transducer is submersible type encased in water proof casing. The pressure range is 1000 kPa ± 0.1%. This transducer monitors the pressure at the formation downhole.

d) Water Pump: The water pump is having a maximum capacity of 151 l/min, attached with regulating valve and liquid-filled pressure gauge. The pump is attached to a 30 HP 3-phase electrical motor operating on 360 VAC to 440 VAC, 50 Hz.

e) Injection Line: 100 m length 31.75 mm aluminium quick connect insertion/ injection rods.

f) Inflation Line: Nylon ¼" dia up to a depth of 110 m. The line is used to inflate the packer systems by nitrogen gas.

The schematic diagram of the double packer permeability system is shown in Figure 7.20 (See Plate 16).

7.9.4 Test Procedure

The cores for the testing are logged and suitable depths for the permeability testing are selected. The test interval (1m perforated pipe) with the double packer assembly is lowered at the desired depth and both the packers are inflated using nitrogen gas. The desirable peak pressure is achieved at each test interval going through low and medium pressure stages. The flow rate is manipulated to keep the pressure constant for 10 minutes at the desired pressure. The pressure is then lowered in two stages to zero.

The pressure-time curve is continuously plotted on analog strip chart recorder.

The pressures used for the results vary with depth and are taken from the relationship:

a) Low pressure "a" in p.s.i. = 0.4 X depth in ft (max. = 50 psi)

b) Medium pressure "b" in p.s.i. = 0.7 X depth in ft (max. = 100 psi)

c) Peak pressure "c" in psi = 1.0 X depth in ft (max. = 150 psi)

7.10 DETERMINATION OF DEFORMBILITY OF ROCK BY PLATE LOAD TEST

Rock masses are heterogeneous and usually discontinuous assemblages of material. As a result the scale of an experiment to some extent determines the validity of the experiment. Comparisons of in situ field tests with laboratory tests results on the same rock show that laboratory tests without fail overestimates the deformability behaviour of the rock masses to a factor of 5 to 15.

The main reasons for this discrepancy are as follows:

a) Presence of joints and bedding,

b) Micro cracks in apparently massive rock, and

c) Localised altered or weathered zones inside the rock masses.

Laboratory specimens are generally taken from rock of good quality without any discontinuities. There is no reliable method of predicting in advance the overall deformation behaviour of the rock mass from the results of laboratory tests. Therefore in-situ field tests for an economical and safe structural design of large and heavy construction are necessary.

The plate-bearing test is one of the most common methods to determine the deformability of rock mass in situ. In this method, a load is applied to a specially prepared flat surface by means of a rigid or semi-rigid finite plate and measuring the deformation at any convenient point within the rock mass. Thus rock modulus can be calculated using the relationships developed depending on the shape of the loading plate and nature of the rock.

Although rock is neither homogeneous nor elastic, it is customary to interpret test results on the basis of the theory of elasticity. Justification of such a procedure rests on the fact that at moderate load rates the stress-strain relations are roughly linear. The accuracy of the determination of the elastic recovery qualities of the rock mass is dependent on the duration of the recovery period in each test.

Data gathered during the test are plotted to provide various cross plots between deformation versus time, pressure and depth of measurements. These plots are used to analyse elastic rebound and permanent set characteristics of the rock.

7.10.1 Application of Plate Load Test

Plate load tests find the following applications

a) These tests are required during the design of large underground excavations since such tests provide the rock mass deformability.

b) Estimation of the grouting efficiency

c) Estimation of rock anisotropy.

d) Estimation of rock behaviour during frequent loading and unloading process.

Both modulus of elasticity and modulus of deformation can be calculated with the same formula as follows:

$$E \text{ or } D = (1- u^2) * \Delta F / \Delta S$$

where,

E: Modulus of elasticity (kgf /cm^2)

D: Modulus of deformation (kgf /cm^2)

u = Poisson's ratio (0.2 for hard rock and 0.25 to 0.3 for soft rock)

Δ F = increased load in a section of load-displacement curve (Kgf = ton/1000)

Δ S = increased displacement for the same section as above (cm)

If a gradient of a tangential line of stress-displacement curve for the peak load is placed in the place of $\Delta F / \Delta S$, the formula will give a modulus of elasticity. If a gradient of a line enveloping the stress-displacement curves of the initial loadings is used for

$\Delta F / \Delta S$, it will give a deformation modulus.

7.10.2 Site Preparations

The areas for testing are carefully selected and all the loose rocks are removed using chipping hammers and drills. An area with diameter of 1.50 m (slightly larger than 2 times the diameter of the test pad, which is 55 cm) is prepared at each site to reduce the restraining influence of adjoining rock. Two instrumentation holes of diameter 76 mm each are core drilled into the prepared test surface keeping the up hole as coaxial as possible with the down hole.

Two concrete pads (one each at top and bottom) having diameter of 55 cm and thickness of 12.70 cm are prepared at each site with reach mix of quick setting cement and aggregate and allowed to cure for twenty eight days. The pads are made flat and parallel to each other.

Site preparation specifications for Plate Load Testing are depicted in Figure 7.21 (see Plate 16).

7.10.3 Equipment to be Deployed

a) Normal Loading Equipment

(i)	Bearing Plate	:	505 mm Dia.
(ii)	Ram Jack Base Plate	:	618 mm
(iii)	Ram Jack Buttons	:	3 Nos
(iv)	Extension Columns	:	to suit 1.5 to 3.0 m tunnel height
(v)	Ram Jacks	:	Capacity 100 tonnes
(vi)	Hydraulic Pump	:	10,000 psi operation
(vii)	Hoses with manifold	:	20 m length

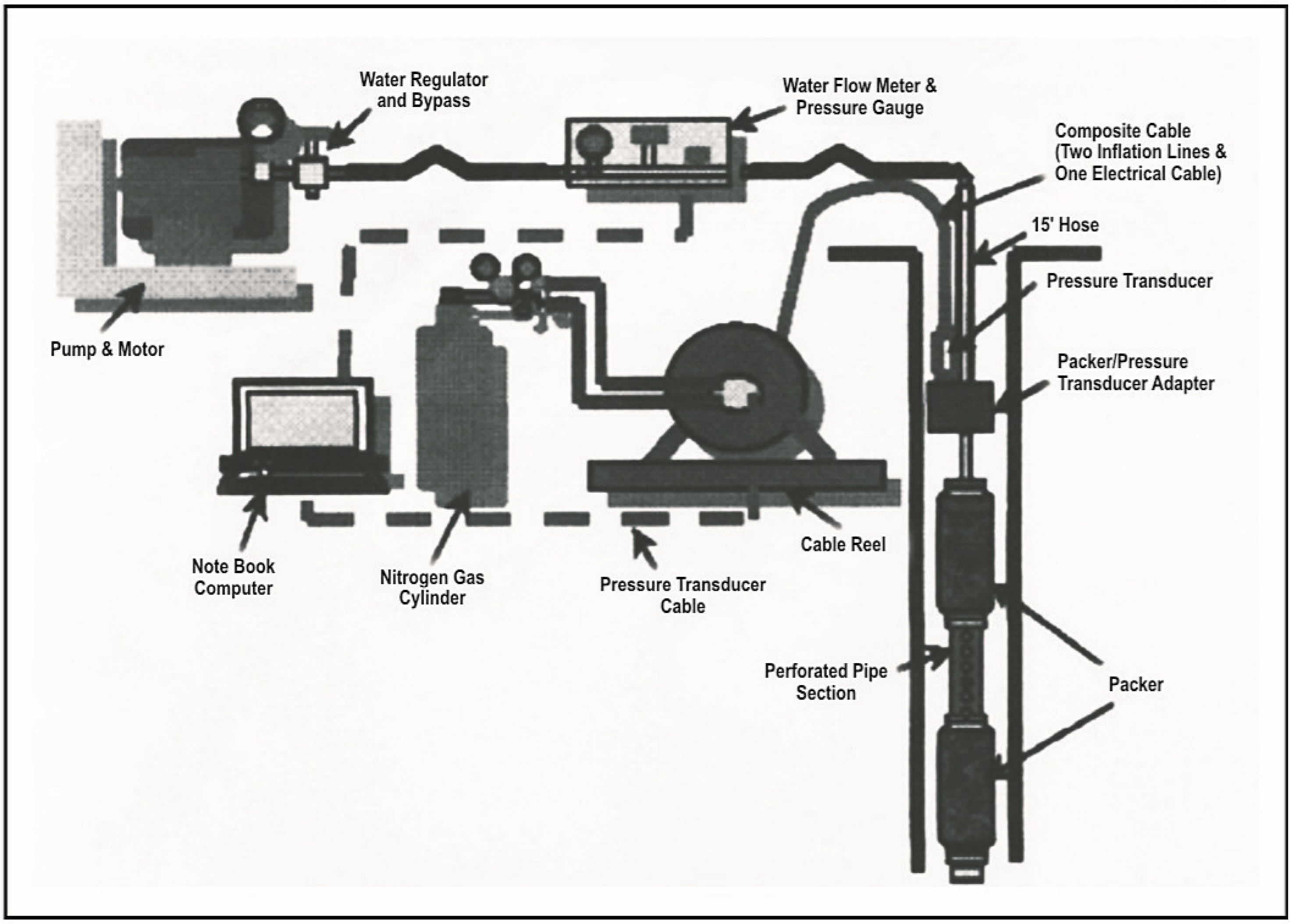


Figure. 7.20 - Schematic Diagram for Double Packer Permeability Test

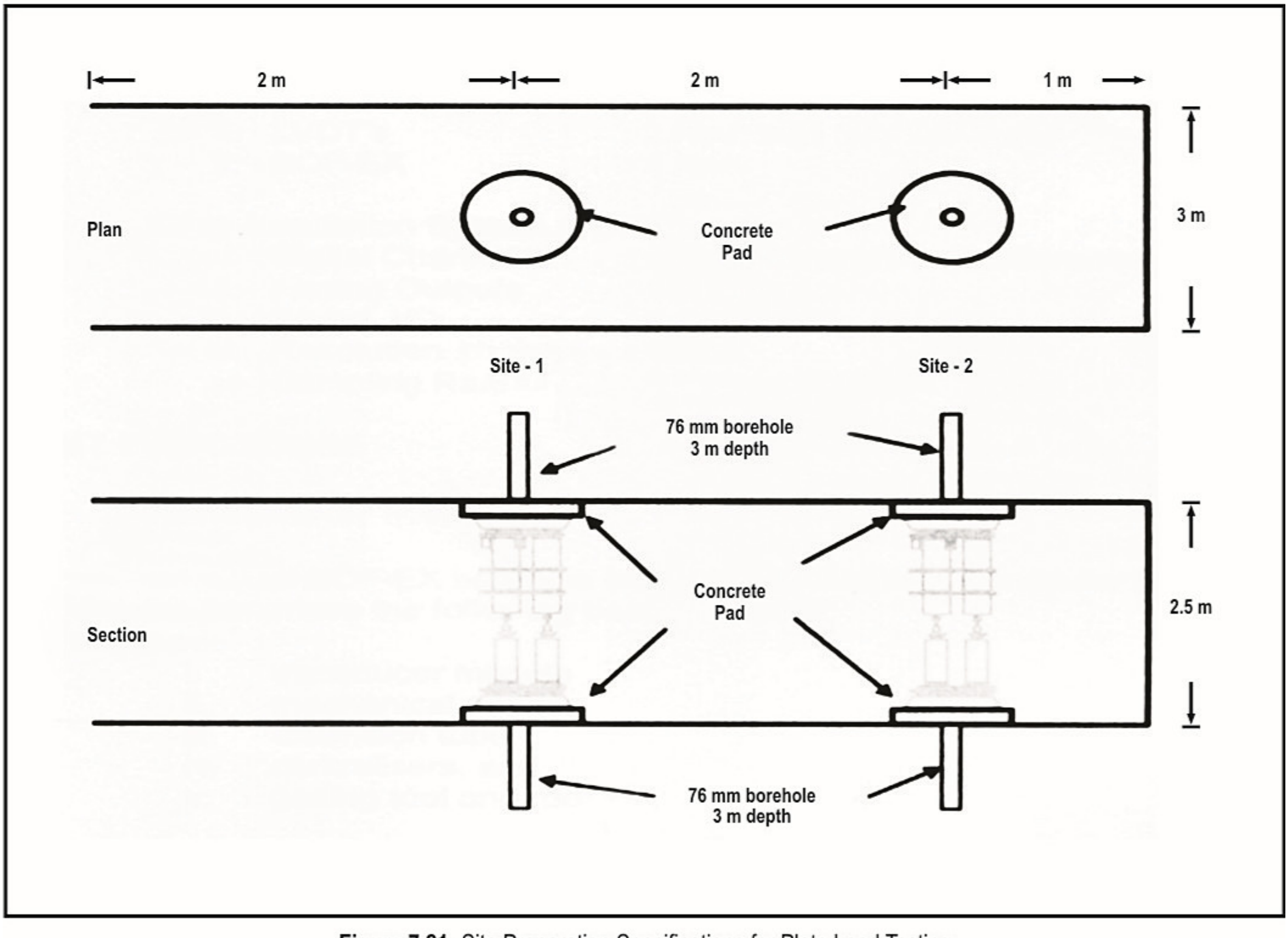


Figure 7.21 -Site Preparation Specifications for Plate Load Testing

b) Deformation Measuring Equipment

(i)	LVDTs	:	2 Nos with 12V DC input
(ii)	BOF-EX	:	4 Nos

c) Data Acquisition System (PCMCIA Card)

(i)	Digital Channels	:	16 single-ended or 8 differential
(ii)	Analog Outputs	:	4 Channels
(iii)	Digital I/O	:	8 Channels
(iv)	Resolution	:	12- bit
(v)	Sampling Rate	:	5 kS/s to 500 kS/s

d) Extensometer Installation

The model BOF-EX borehole extensometer system is modular in design and comprises the following basic elements:

(i) transducer module

(ii) mechanical anchor

(iii) extension tube

(iv) centralisers, and

(v) setting tool and rods

The anchor is composed of a disk supporting three pads spaced at 120^{o} and a central cylinder housing a jacking screw. The anchor is designed to fit boreholes with a nominal diameter of 76 mm. One end of the anchor is terminated with a bayonet connector and a screw with a hexagonal head. The anchor is expanded in the hole using a string of concentric setting rods. The inner rod with the hexagonal head is turned clockwise to actuate the screw jack. The other 'end of the anchor has a short threaded cylinder to attach the extension tubes or transducer module. The transducer module is attached to one or more extension tubes in order to span the required distance between anchors. The centraliser is used to centre and support the transducer modules and extension rods.

After determining the depth of the deepest anchor the bayonet head is connected on the installing tool to the anchor. The outer installing tube is then connected to the anchor installation tool and the whole assembly is inserted into the borehole. The anchor is then expanded and installed with the help of inner rods. Once anchor is installed to its desired depth the inner and the outer assemblies are withdrawn. The transducer module is then Installed. The other anchors and transducer assemblies are installed likewise.

Installation of the Plate Loading Equipment

After installation of all the extensometers, a particle board of 55 cm diameter with a hole at the centre is placed on the lower concrete pad (for horizontal testing on one of the concrete pads). On the particle board the load distribution plate 50.50 cm diameter is placed with ram jack base plate. Three numbers of ram jacks each of 100 tons capacity are placed above the ram jack base plate with the spherical seats. The extension columns are placed above another up to the top concrete pad leaving enough space to accommodate a bearing plate, a load distribution plate of the same diameter as the bottom one and a particle board.

All the extensometer cables and the electric pump's interface are connected to the PCMCIA card (digital and analog inputs) which in turn is hooked to the Pentium computer. Support software Genie is used for data acquisition, process and real time multiple tasking. The electric pump's manifold with hoses are then connected to the three jacks. Schematic diagram of Plate load testing set-up and Erection of Plate loading equipment to conduct Deformability test are shown in Figures 7.22 & 7.23 (See Plate 17).

Testing Programme

After installation of all the components and checking all the mechanical and electronic components, zero readings of all the extensometers are taken at zero pressure. The pressure is raised step by step by the electric pump and same is transmitted by the loading plate of 50.50 cm diameter to the rock through 55 cm diameter concrete pad of 12.70 cm thickness. The pressures at each stage is maintained using a controller valve. The pressure and deformation readings are automatically recorded on the hard disk of the computer continuously. The data after every test are stored into floppy for analysis purpose.

The Plate loading test is conducted by applying and changing vertical pressure on the steel loading plate at the bottom, and recording vertical displacement of the plate by four dial gauges, in the way as described below:

The pressure is raised by four steps, that is 14 kgf/cm^2, 28 kgf/cm^2, 42 kgf/cm^2 and 56 kgf/cm^2.

(i) Firstly the pressure is raised up to 14 kgf/cm^2 at the rate of 2.8 kgf/cm^2 every minute. Displacements are read by the LVDT at every 2.8 kgf/ cm^2 of rise of the pressure, viz., 0 kgf/cm^2, 2.8 kgf/cm^2, 5.6 kgf/cm^2, 8.4 kgf/cm^2 11.2 kgf/cm^2 and 14 kgf/ cm^2. The peak pressure of 14 kgf/ cm^2 is maintained for 10 minutes, reading the dial gauge every two minutes. Then the pressure is lowered gradually to zero at the rate of 7 kgf/cm^2 every minute, recording the displacement for every 7 kgf/cm^2 of load decrease. The zero load is maintained for 10 minutes, with the dial gauge being read every two minutes.

(ii) After the elapse of 10 minutes of zero pressure, the pressure is raised up to 28 kgf/cm^2 with the same rate of pressure increment and the LVDT reading for every 2.8 kgf/cm^2 of raise in the pressure is read. The pressure is maintained at 28 kgf/cm^2 for 10 minutes, and is lowered to zero at the rate of 7 kgf/cm^2 every minute, where the displacement is recorded every 7 kgf/cm^2 of descent. Then, the zero pressure is maintained again for 10 minutes. When the pressure is kept constant, the LVDT readings are read every two minutes. All procedures are repeated to the case of the peak of 14 kgf/cm^2 except that the peak pressure is different.

(iii) Next, the pressure is raised to the peak of 42 kgf/cm^2 and lowered back to zero in the same manner.

(iv) In the fourth cycle of increasing and decreasing the pressure, the peak is 56 kgf/cm^2 and the tests are advanced following the same method as above.

Data Acquisition System (PCMCIA Card)

(i)	Digital Channels	:	16 single ended or 8 differential
(ii)	Analog Outputs	:	4 Channels
(iii)	Digital I/O	:	8 Channels
(iv)	Resolution	:	12- bit
(v)	Sampling Rate	:	5 kS/s to 500 kS/s

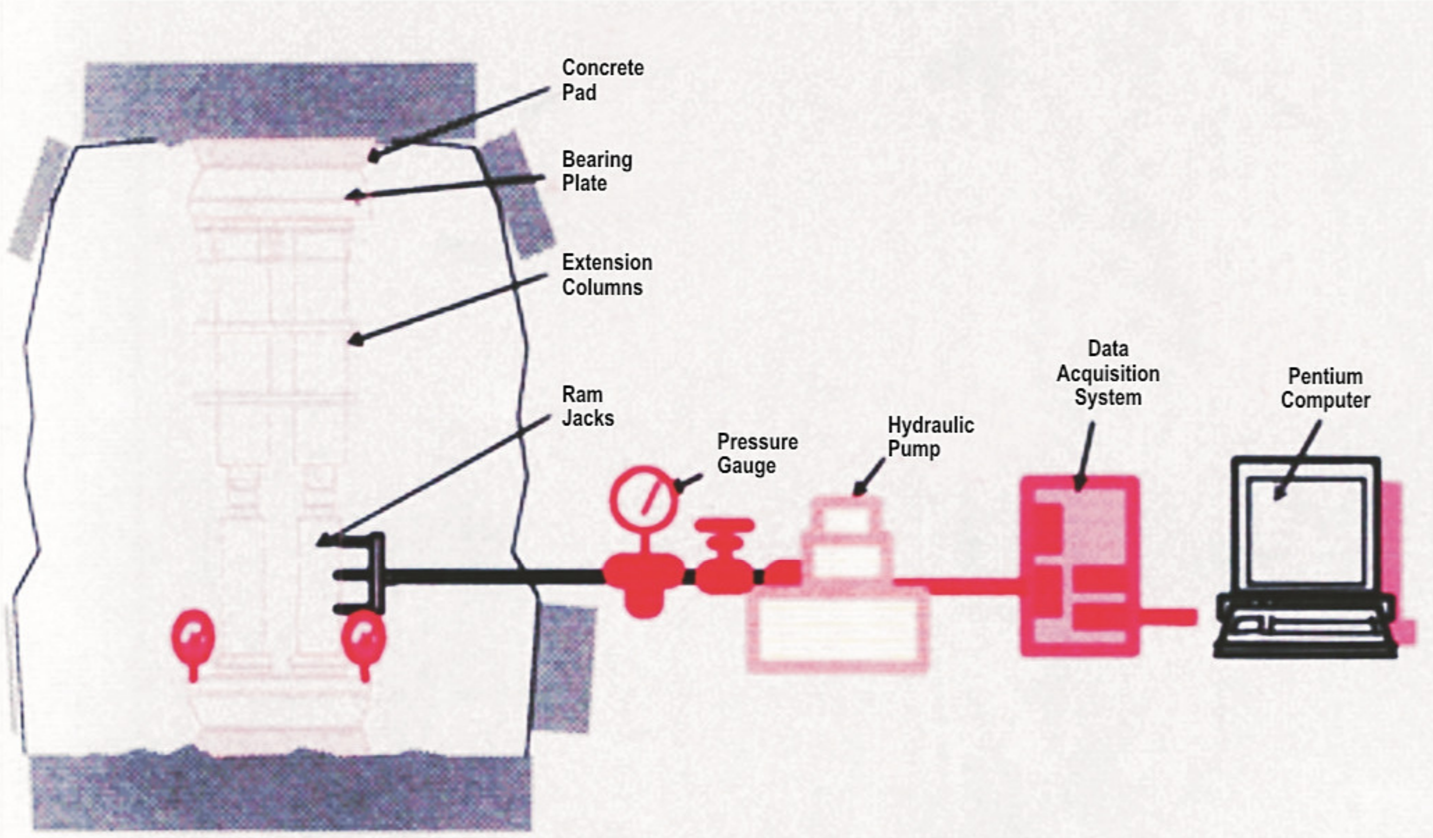


Figure 7.22 - Schematic Diagram of Plate Load Testing Set-up

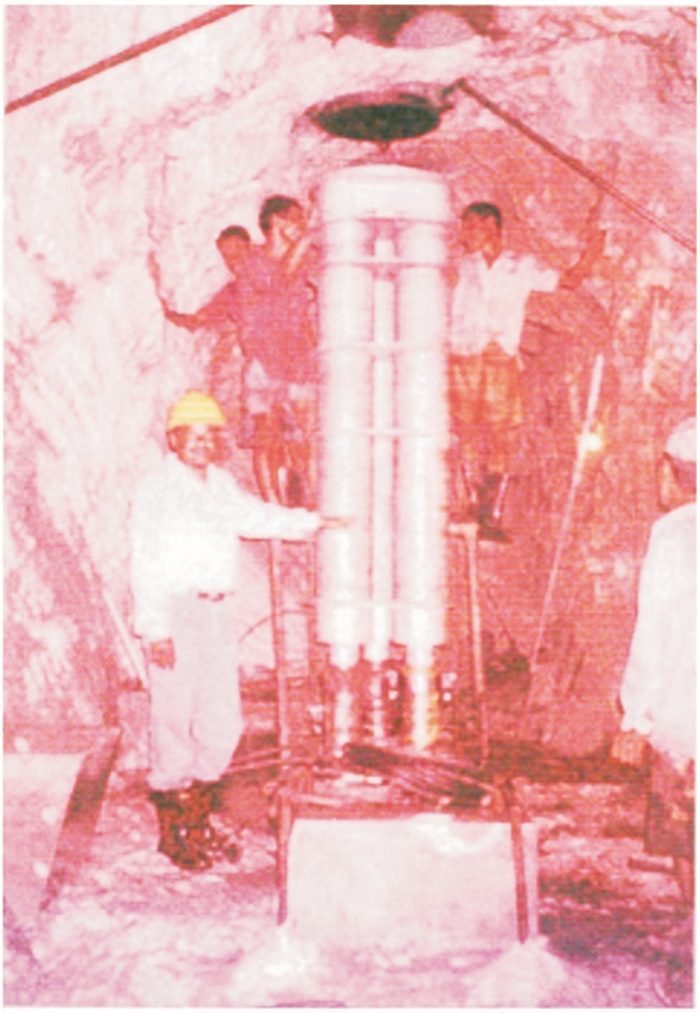


Figure 7.23 - Erection of Plate Loading Equipment to Conduct Deformability Test

7.11 DETERMINATION OF DISPLACEMENT BY TAPE EXTENSOMETER

The tape extensometer quickly and accurately measures changes in distance between two reference points in any direction. Typical applications are the measurement of:

- Mine roof sag and tunnel closure
- Displacement of unstable slopes and structures
- Magnitude of behaviour in ground during excavation
- Radial movement and convergence of tunnels, shaft linings and caverns, support deformation of excavations. The procedure of using tape extensometer is shown in Figure 7.24.

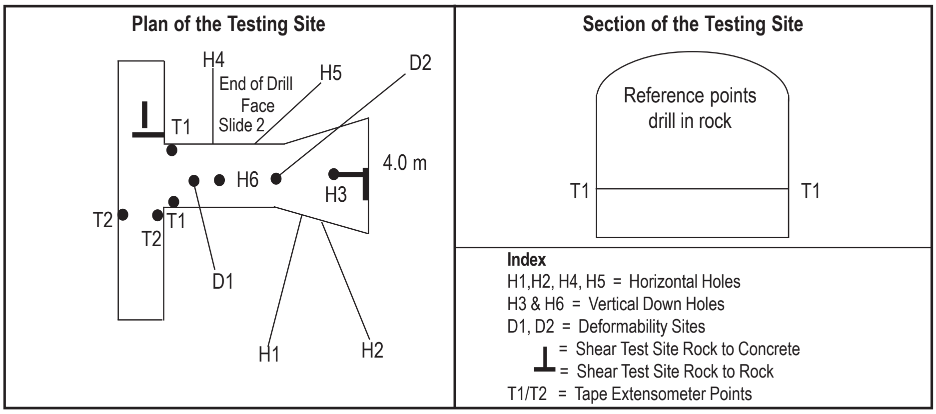


Figure 7.24 Plan and Section of Testing Site

Steel Tape : A steel "gauging" tape is used to span the distance between the two reference points. The tape is graduated in English or metric units and is perforated at 2-inch or 50 mm intervals.

Hooks : The free end of the tape has a snap hook, which is attached to one reference point. The tape is then unreeled until the operator can attach the hook on the instrument body to a second reference point.

Nose & Pin : The instrument nose has a slot for the tape. The operator slips the tape into the slot and engages a pin in the nearest hole in the tape. The pin provides a positive hold on the tape so it can be tensioned.

Tension Collar & Index Marks : The tension collar is rotated to apply tension to the tape. When the index marks are aligned, the tape is correctly tensioned.

Sliding Scale and Dial : A tape extensometer measurement is actually the sum of three readings: the reading from the tape, the reading from the sliding scale and the reading from the dial.

7.11.1 Features

- Lightweight and rugged design
- Rapid measurement and one-man operation
- Optimised design to protect steel tape from damage
- Complete steel version available on request

Tape extensometer is a portable instrument to measure the displacement between pairs of reference studs grouted into shallow drill holes in the structure of excavation. The studs are permanently fixed to provide a precise fixing point at the surface of the structure. Demountable extension studs are available for fixing in location where, for instance, additional shotcrete is applied to the surface to be monitored.

The tape extensometer basically consists of a steel survey tape with punched hole loaded on a reel fixed to the body of the instrument. It incorporates a tensioning mechanism for the tape as well as dial /digital indicator-based distance measuring system. Two hooks are provided—one at the movable extremity on the tape and the other on the reel frame. Tensioning of the tape to a predetermined load is easily done by rotating large knurled collar until two reference lines are precisely aligned.

7.11.2 Installing Reference Points

Reference points are stainless steel eyebolts that are threaded into groutable or expansion anchors. Reference points may also be bolted to the structure. Reference points are positioned to reveal the magnitude and direction of movements. The drawings below shows typical locations for reference points. Since each site has unique conditions, the pattern of your reference points may not resemble those illustrated.

The hook & eye bolt system can accommodate almost any angle of the tape. However, it is important to protect the points once they have been installed, since any change in the position or the condition of the points will affect the repeatability of the system.

Using Groutable Rebar Anchors in Rock or Concrete

Parts Required

Eyebolts (1/4-20 thread). Be sure to have a spare eyebolt for installation and testing of the anchor.

7/16 inch lock nuts (1/4-20 thread).

Stud (1/4-20 thread).

- Rebar anchor (length depends on competence of rock/concrete).
- Non-shrinking grout. Or see next page for use of epoxy cartridges.

Tools Required

Rock drill and bit capable of drilling 3/4" or 1" diameter holes slightly deeper than the length of the rebar anchor.

- Compressed air to clean hole - 7/16" open-end wrench - Adjustable wrench

Instructions

Select and mark locations for reference points.

Drill 3/4" or 1" hole to depth approximately 1/2-inch deeper than length of anchor. Clean debris from hole with compressed air.

Grout anchor in hole using a suitable mixture of non-shrinking, "underwater" grout. If necessary, pack the hole to prevent grout seepage.

When the grout is hard, thread a spare eyebolt into the anchor and pull test the anchor to 30 lbf (max). If there is no measurable movement, the anchor is installed satisfactorily. Remove the spare eyebolt.

Thread a lock nut onto the eyebolt, and then thread the eyebolt into the anchor. If it is difficult to turn the eyebolt, use an adjustable wrench (closed across the circle of the eyebolt) for additional leverage. Important: Do not use a screw driver for additional leverage, since any implement inserted through the eyebolt may deform the eyebolt and make it difficult to obtain repeatable readings in the future. Hold the eyebolt in position with the adjustable wrench (closed across the eyebolt circle) and tighten the lock nut with the 7/16" open-end wrench.

7.11.3 Taking Readings

A. Allow sufficient time for the instrument to adjust to the ambient temperature at the measurement station.

B. Disengage the tape from the pin and slide it out of the nose slot. Check that the tape crank is ready and that the tape can be unreeled.

C. Rotate the tension collar until the sliding scale reads 1" or 25 mm. To measure only convergence, the collar would need to be rotated until the sliding scale reads 2" or 50 mm. Do not rotate the collar further or the dial may be damaged.

Hook onto Reference Points

1. Attach the snap hook to a reference point.
2. Carry the instrument towards the second reference point, allowing the tape to unreel as as it is moved.
3. Hook the instrument body onto another second reference point.
4. Wind up the slack in the tape.
5. Slide the tape into the instrument nose.
6. Pull back the spring clip and fit the pin into the nearest hole. Try to choose the hole that makes the tape as tight as possible. With tape spans greater than 15 ft or 5 meters, it must be pulled to cause tension till the appropriate hole is found.
7. Unwind a little tape from the reel so as to facilitate free access to the tension collar.

Tension the Tape

The tape must be tensioned properly so that repeatable measurements could be obtained. Rotate the tension collar to adjust the tension.

1. Support the instrument with one hand and point the nose of the instrument directly at the opposite reference point.

2. Rotate the tension collar to adjust the tension. Look at the pointer on the dial to determine which direction to rotate the collar. The pointer moves counter-clockwise when you increase the tension; clockwise when you decrease the tension.

3. As you make fine adjustments to the tension, move the nose of the instrument up and down.

When the nose is not pointing directly at the reference point, the index marks should show over-tension.

The index marks must not show under-tension in any position. After the index marks are aligned, you can let the instrument could be allowed to hang unsupported.

Do not allow the tension collar to move. The instrument is now ready to be read.

Reading the Instrument

A tape extensometer reading is actually the sum of three readings—the reading from the tape, the sliding scale, and the dial (see Figure 7.25—Plate 18).

7.11.4 Specifications

Measuring range	up to 30 m
Resolution	0.05 mm
Repeatability	+ / - 0.1 mm
Material	Anodising Aluminium

7.12 DETERMINATION OF IN SITU SHEAR STRENGTH OF ROCK BY DIRECT SHEAR TEST

The shear strength of rock mass is one of the most important parameters used in design. The shear strength parameters cannot be predicted on the basis of case histories and on any classification system. Therefore, the feasibility of conducting in situ shear tests cannot be avoided due to variation of rock mass properties at different sites.

The best way of evaluating shear strength parameters is by conducting in situ shear test. The procedure of conducting the in situ shear tests is described in ISRM (1974) and IS 7746-(1975). The shear strength of concrete to rock depends upon number of factors, such as, strength of concrete, strength of rock, saturation, rate of loading, rate of shearing etc.

7.12.1 Application of In situ Shear Test

Typical applications include

- Shear strength of foundation rock of dams
- Shear strength of rock to concrete interface for Dam foundations
- Design of underground excavations

This test measures peak and residual direct shear strength as a function of stress normal to the sheared plane.

Peak direct shear strength - the maximum shear stress in the shear stress versus displacement curve.

Residual shear strength – the shear stress at which no further rise or fall in shear strength is observed with increasing shear displacement.

Shear strength determination of rock mass should preferably comprise at least five tests on the same test horizon with each specimen tested at a different but constant normal stress.

Displacement readings are averaged to obtain values of mean shear and normal displacement Δs and Δn. Lateral displacements are recorded only to evaluate specimen behaviour during the test. For the evaluation and interpretation of the shear testing data, shear and normal stress are computed as follows: (Ref: Suggested methods for determining Shear Strength, ISRM Standard)

$$\text{Shear stress} = \tau = P_s/A = (P_{sa} \cos \sigma)/A \quad (1)$$

$$\text{Normal stress} = \sigma n = P_n/a = (P_{na} + \sin \sigma)/A \quad (2)$$

where, P_s = total shear force

P_{na} = total normal force

P_{sa} = applied shear force

P = applied normal force

σ = inclination of the applied shear force to the shear plane

(if, $\sigma = 0$, $\cos \sigma = 1$ and $\sin \sigma = 0$)

A = area of the shear surface overlap

7.12.2 Site Preparation

The sites are selected inside the test drift. All the slacked rocks are removed and the surface on tight rock is trimmed to nearly flat and horizontal. Tight rock surfaces are exposed and cleaned. Three blocks of concrete of the sizes 700 mm x 700 mm x 350 mm are made over the specially made and cleaned rock surface. The part of inclined surface of concrete block, protruding to one side is only meant for convenience of loading for shear. Thus the bottom of this inclined part of the block is prevented from contact with the rock surface by laying sand when concrete is placed.

A channel approximately 20 mm deep by 80 mm wide is cut around the base of the block to allow freedom of shear and lateral displacements. The site preparation specifications, which are necessary to conduct in situ shear test as well as the plan and section view indicating the specifications of the specimen for shear testing are depicted in the Figures 7.26 & 7.27 (See Plate 18).

7.12.3 Equipment to be Deployed

a) Normal Loading Equipment

(i)	Steel plate	:	700 mm x 700 mm x 350 mm
(ii)	Steel Roller	:	500 mm x 500 mm
(iii)	Ram Jack Base Plate	:	505 mm dia
(iv)	Ram Jack Buttons	:	3 Nos.
(v)	Extension Columns	:	to suit 1.5 to 3.0 m tunnel height
(vi)	Ram Jacks	:	Capacity 100 tonnes
(vii)	Bearing Plate	:	505 mm dia
(viii)	Hydraulic Pump	:	10,000 psi operation
(ix)	Hoses with manifold	:	20 m Length

b) Shear Loading Equipment

(i)	Shear Loading Jack	:	300 Ton Capacity
(ii)	Load Cell	:	300 Ton Capacity
(iii)	Hydraulic Pump	:	10,000 psi operation
(iv)	Hose	:	20 Mts

c) Deformation Measuring Equipment.

(i)	Dial Guages	:	0.01 mm LC
(ii)	LVDT's	:	With 12 V DC input

d) Data Acquisition System (PCMCIA Card)

(i)	Digital Channels	:	16 single ended or 8 differential
(ii)	Analog Outputs	:	4 Channels
(iii)	Digital I/O	:	8 Channels
(iv)	Resolution	:	12-bit
(v)	Sampling Rate	:	5 Ks/s to 500 Ks/s

7.12.4 Test Procedure

a) Installation of the Normal Loading Equipment

A plate of size 700 mm x 700 mm x 350 mm is placed on the concrete block. A roller system is then placed on the top part of the plate. A plate with one side square and the other part circular of 50.50 cm diameter is then placed over the roller bearing system with ram jack base plate. Three numbers of ram jacks each of 100 tons capacity are placed above the ram jack base plate with the spherical seats. The extension columns are placed one above another up to the top leaving enough space to accommodate a bearing plate, a load distribution plate of the same diameter as the bottom one and a particleboard. All the jacks are connected with an electrical pump with a toggle switch for producing a constant normal load. A pressure transducer attached to the electric pump is connected with the online data acquisition system.

b) Installation of the Shear Loading Equipment

A 300 tons jack is placed inclined to the shear surface of the concrete (concrete-rock interface) block. The other side of the jack is placed against concrete pad especially made against the wall of the drift to receive the reaction from the wall. A 300 tons load cell is installed in between the rock block and the jack meant for shear loading. The shear loading is produced by a hand pump system

c) Installation of the Deformation Measuring Equipment

A measuring frame with four dial gauges serve to measure the settlement of the test body by the normal load. The four dial gauges that are mounted on the frame, measure the shear displacement of the test block caused due to shearing force applied. Four more dial gauges are mounted on the frame which monitor the lateral displacement of the block during shearing. The measuring frame is made of rigid galvanised steel pipes, which are anchored in the rock formation at an appropriate distance from the test location.

Schematic diagram of shear testing set-up is shown in Figure 7.28 (see Plate 19). In situ shear testing at one of the sites is shown in Figure 7.29 (see Plate 19). Shear testing set up indicating Shear Jack details at one of the sites is shown in Figure 7.30 (see Plate 20). In situ shear testing at one of the sites (showing Dial gauges and LVDT) is shown in Figure 7.31 (see Plate 20).

d) Loading Pattern for Shear Test

After setting up of the equipment, the following loading pattern would need to be followed:

Loading for the shear test by jacks consists of vertical loading and inclined loading. For each test block, the vertical load by a jack will be kept constant for all the course of the test and the inclined load on the inclined face of the block will be increased by step until the rock at the bottom is sheared. Shearing of three blocks on the same condition of rock will be made one set of shear test, where three different levels of the constant vertical load will be used for those three blocks, that is 24.5 tons (5 kg/cm^2), 29.25 tons (7.5 kgf/cm^2) and 49 tons (10 kgf/cm^2). A block will be sheared under a constant vertical load of 24.5 tons, and another is under a constant vertical load of 29.25 tons, and so on.

e) Preliminary Vertical Loading

Preliminary vertical loading will be made initially for contact adjustment of the equipment with the test block. Preliminary vertical load will be raised up to 24.5 tons (5 kg/cm^2) by the rate of 12.25 tons/min

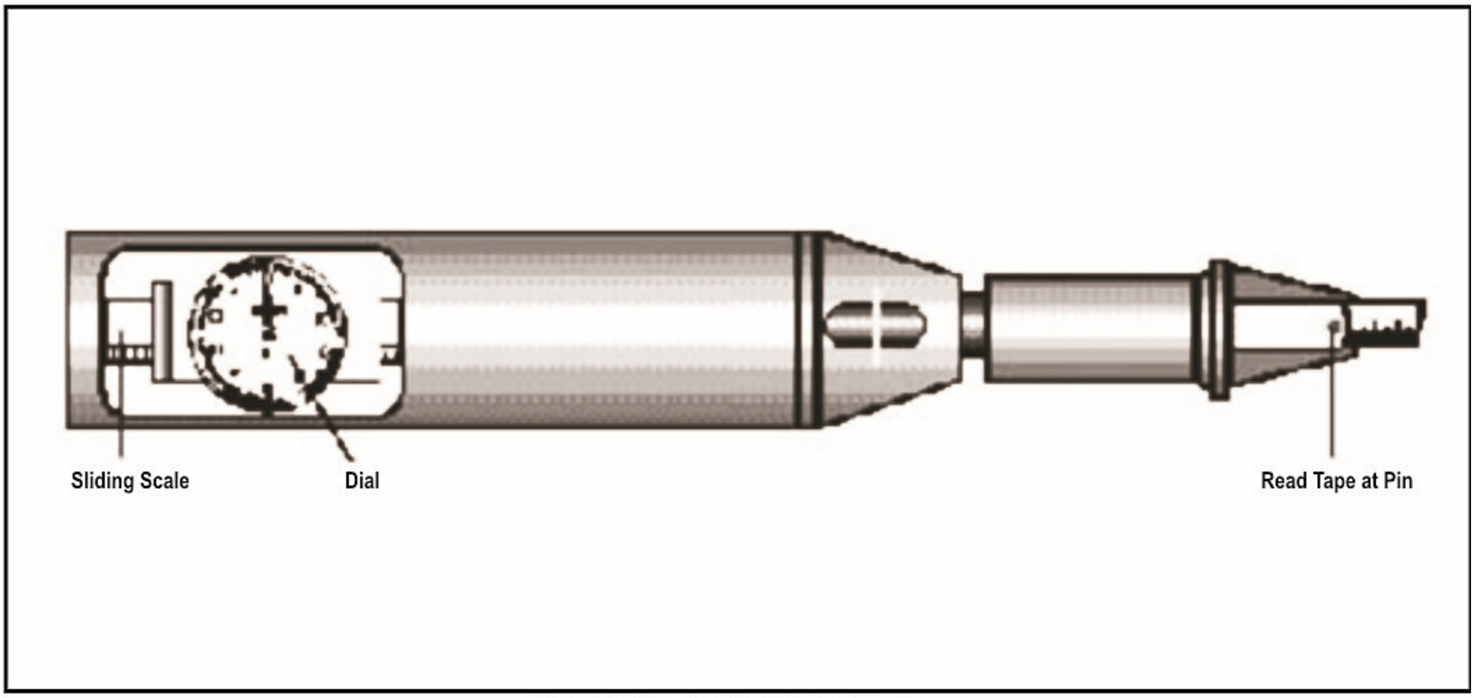


Figure 7.25 – Reading the Instruments

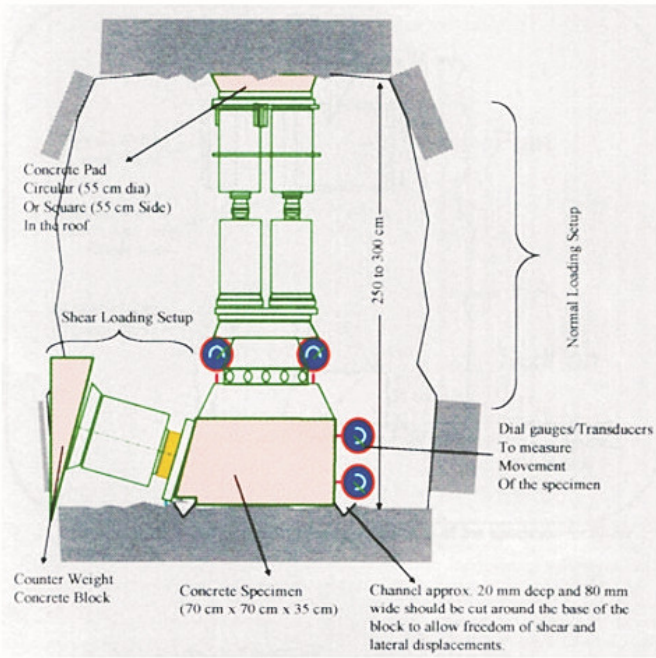


Figure 7.26 - Section view of the site preparation specifications for conducting shear test.

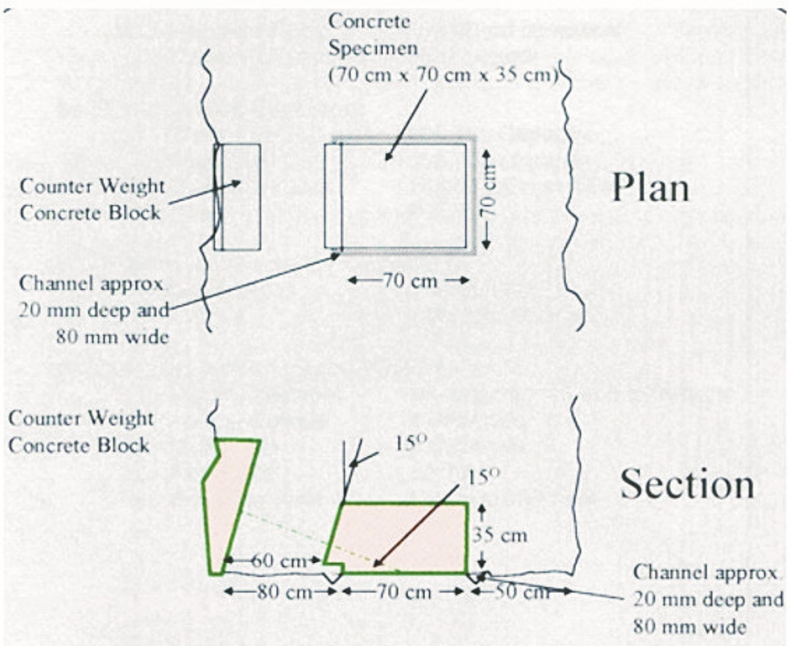


Figure 7.27- Plan and Section View Indicating the Specifications of the Specimen for Shear Testing

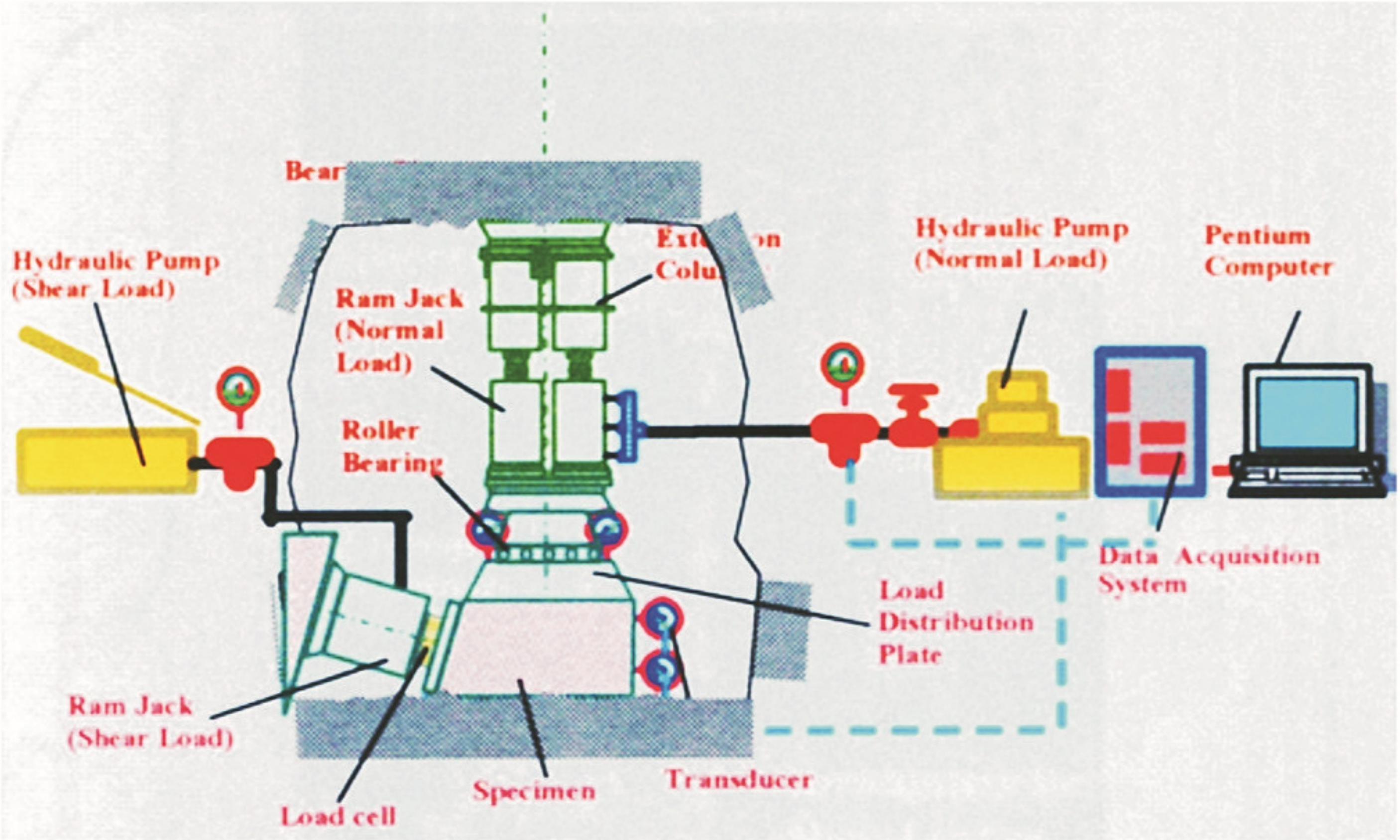


Figure 7.28 - Schematic Diagram of Shear Testing Set-up

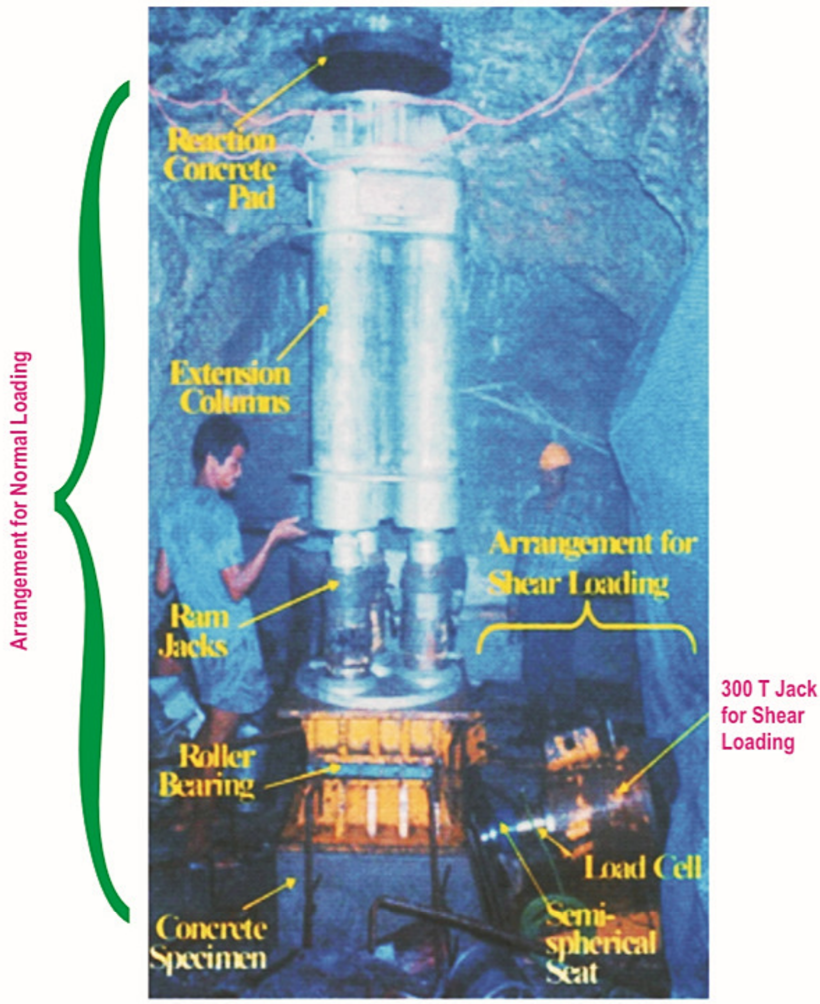


Figure 7.29 - In situ Shear Testing at One of the Sites

Figure 7.30 - Shear Testing Set-up Indicating Shear Jack Details at on of the Sites

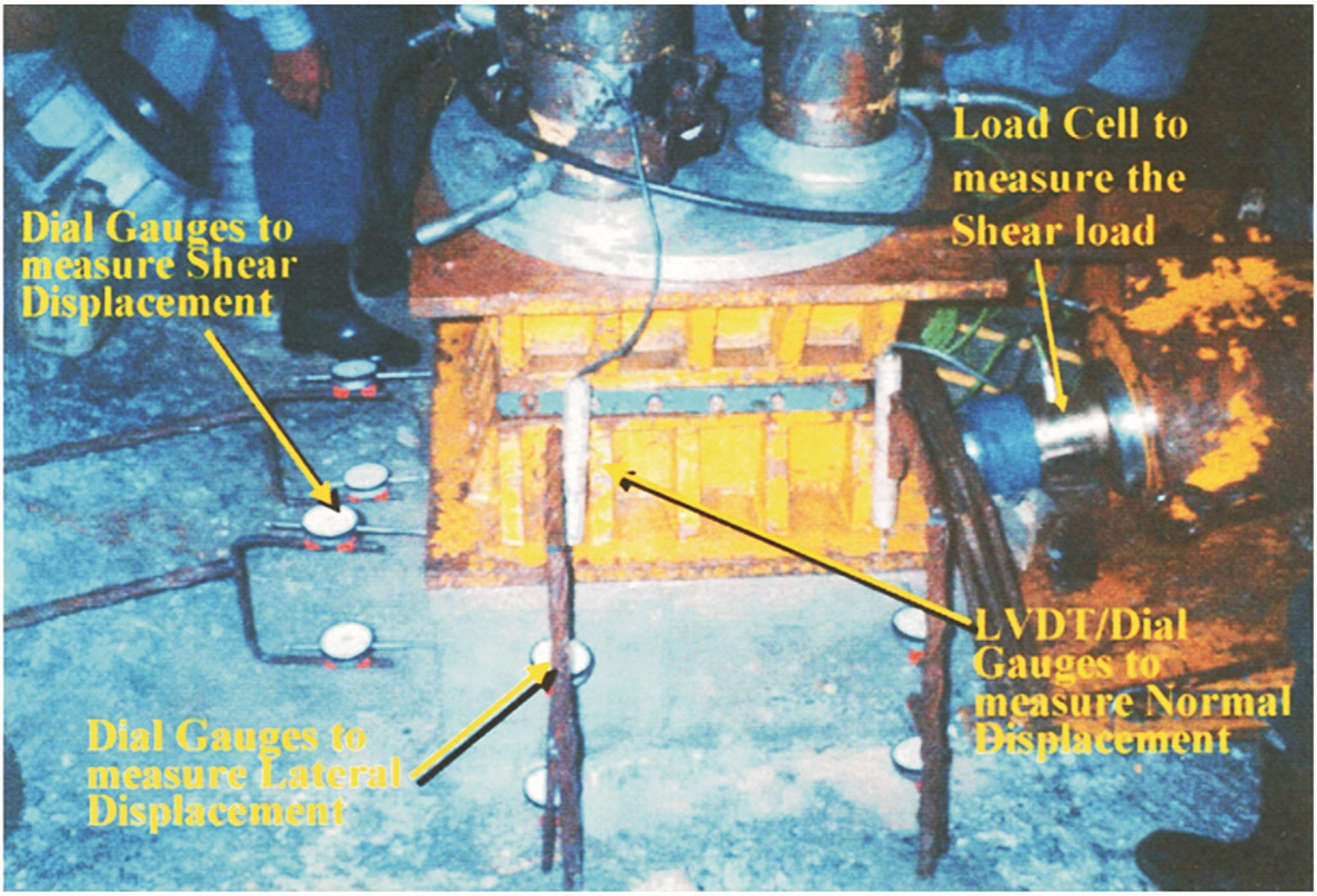


Figure 7.31 - In situ Shear Testing at one of the Sites (Showing Dial Gauges and LVDT)

(2.5 kgf/cm^2/min) and the LVDTs at every rise of 12.25 tons in 3 minutes are read. The load is suspended for 5 minutes at 24.5 tons, and the displacements are noted at the beginning and at the end of 5 minutes. The load is reduced to zero in the same rate as in the raising course and the displacements are read at 9 tons and zero. The same pattern of loading and unloading is repeated once again before a constant vertical load is achieved, whatever the constant load might be.

f) Inclined Loading for Shear

Maintaining the vertical load constant, the inclined load on the inclined surface of the test block will be increased by step of 6.8 tons (1.39 kgf/cm^2), by the rate of 6.8 tons in 3 minutes. The displacement in every step is observed with the ten dial gauges installed, during the time of 5 minutes to keep the shear load constant.

The inclined load will be increased in this way until the rock at the bottom of the test block is sheared. Occurrence of the shear is known by sudden increase of the horizontal movement and sudden fall of the load. The normal and shearing forces along with vertical and horizontal displacements will be recorded during the shearing. The observations will be continued after the failure of the block for studying the residual shear strength parameters.

Measurement of Change in Stress

For measuring the change in stress in the underground openings vibrating wire stress meters are used. The stress meter measures the change in stress within the rock. It can be used for monitoring underground excavation including shaft, tunnels and in the stopes. For installing stress meter NX borehole is to be drilled in to the rock and after taking the core. The stress meter is installed at a convenient depth (6m, 8m & 10m) depending upon the site conditions. The Figure 7.32 (see Plate 21) illustrates the vibrating wire stress meter capsule.

The stress meter consists essentially of a high strength steel that provide a ring wedge tightly across one diameter of a borehole drilled into the rock. The distortion of the providing ring, caused by changing rock stresses, is measured by means of a vibrating wire. Change in rock stress caused changes in the resonant frequency of vibration of the tensioned wire, and the two are related by means of calibration data supplied with each stress meter.

The stress meter behaves as a rigid inclusion—the calibration varies by a factor of two only if the rock modules varies by a factor of ten.

7.13 CONVERGENCE INDICATOR

Strata monitoring is essential to evaluate the performance of the cable bolted roof. For this, convergence indicator, extensometer and instrumented cables are used. Convergence indicator is a simple instrument consisting of a graduated rod fitted in a pipe. It has a least count of 0.5 to 1 mm, and the telescopic movement is for a length of 2 to 4 m. The measuring points reference ("stations") are metal rods grouted in the roof and floor. Measurements are taken by simply stretching the telescopic rod between the reference points, and reading the graduations on the rod (Figure 7.33).

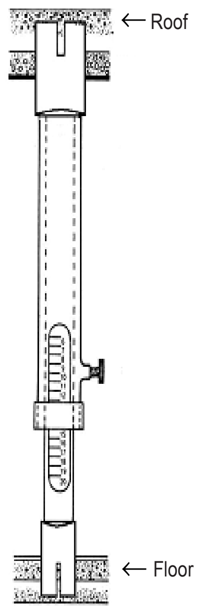


***Figure 7.33:** Convergence Indicator*

7.14 ROCK SUPPORT

The primary objective of a support system is to mobilise and conserve the inherent strength of the rock mass so that it becomes self-supporting. Rock support generally combines the effects of reinforcement, by such elements as dowels, tensioned rock bolts & cables and support with shotcrete, mesh & steel sets which carry loads from individual rock blocks isolated by structural discontinuities or zones of loosened rock. These notes are intended to assist the underground support engineer in choosing the most appropriate and the one that is easiest to install which would impart both reinforcement and support. If possible, the installation of rock support should be carried out as an integral part of the excavation cycle to enhance the self-supporting aspects of rockmass improvement.

The choice of the type of support to be installed in underground excavation depends upon the extent of the zone of loosened or fractured rock surrounding that excavation. A very crude guide to support selection is given in Table 7.1.

Table 7.1: Suggested Support for Various Rock Conditions

Rock Conditions	Suggested Support Type
Sound rock with smooth walls created by good blasting. Low in situ stresses.	No support or alternatively, where required for safety, mesh held in place by grouted dowels or mechanically anchored rockbolts, installed to prevent small pieces from falling.
Sound rock with few intersecting joints or bedding planes resulting in loose wedges or blocks. Low in situ stresses.	Scale well then install tensioned, mechanically anchored bolts to tie blocks into surrounding rock. Use straps across bedding planes or joints to prevent small pieces falling out between bolts. In permanent openings, such as, shaft stations or crusher chambers, rockbolt should be grouted with cement to prevent corrosion.
Sound rock damaged by blasting with a few intersecting planes. Low in situ stresses	Chain link or weld mesh held by tensioned mechanically anchored rockbolts, to prevent falls of loose rock. Attention must be paid to scaling and improving blasting to reduce amount of loose rock.
Closely jointed blocky rock with small blocks ravelling from surface causing deterioration if unsupported. Low stress conditions.	Shotcrete layer, approximately 50 mm thick. Addition of micro-silica and steel fibre reduces rebound and increases strength of shotcrete in bending. Larger wedges are bolted so that shotcrete is not overloaded. Limit scaling to control ravelling. If shotcrete not available, use chain-link or weldmesh and pattern reinforcement, such as, split sets or Swellex.
Stress-induced failure in jointed rock. First indication of failure due to high stresses are seen in borehole walls and in pillar corners.	Pattern support with grouted dowels or Swellex. Split sets are suitable for supporting small amounts of failure. Grouted tensioned or untensioned cables can be used but mechanically anchored rockbolts are less suitable for this application. Typical length of reinforcement should be about ½ the span of openings less than 6 m and between ½ and ⅓ for spans of 6 to 12 m. Spacing should be approximately ½ the dowel length. Support should be installed before significant movement occurs. Shotcrete can add significant strength to rock and should be used in long-term openings (ramps etc.). Mesh and straps may be required in short-term openings (drill-drives etc.)
Drawpoints developed in good rock but subjected to high stress and wear during blasting and drawing of stopes.	Use grouted rebar for wear resistance and for support of drawpoint brows. Install this reinforcement during development of the trough drive and drawpoint, before rock movement takes place as a result of drawing of stopes. Do not use shotcrete or mesh in drawpoints – place dowels at close spacing in blocky rock.

Rock Conditions	Suggested Support Type
Fractured rock around openings in stressed rock with a potential for rockbursts	Pattern support required but in this case some ‘flexibility’ is required to absorb shock from rockbursts. Split sets are good since they will slip under shock loading but will retain some load and keep mesh in place. Grouted dowels and Swellex will also slip under high load but some face plates may fail. Mechanically anchored bolts are poor in these conditions. Lacing between heads of reinforcement helps to retain rock near surface under heavy rockbursting.
Very poor rock associated with faults or shear zones. Rock-bolts or dowels cannot be anchored in this material.	Fibre-reinforced shotcrete can be used for permanent support under low stress conditions or for temporary support to allow steel sets to be placed. Note that shotcrete layer must be drained to prevent build up of pressure behind the shotcrete. Steel sets are required for long-term support where it is evident that stresses are high or that the rock is continuing to move. Capacity of steel sets estimated from amount of loose rock to be supported.

7.14.1 Active Rock Reinforcement

Underground mines use two principal types of rock reinforcement – tensioned mechanically anchored rockbolts and untensioned grouted or friction anchored dowels.

Mechanically Anchored Rockbolts

Mechanically anchored rockbolts are probably the oldest form of rock reinforcement used in underground mining and are still the most common form of rock reinforcement used in Canadian mines. Provided that the rock is hard enough to provide a good grip for the anchor, an expansion shell anchor which is well seated will usually allow a rockbolt to be tensioned to its maximum load-carrying capacity. In fact, if a bolt is overloaded, it usually fails in the threads at either the faceplate or anchor end rather than by anchor slip.

Tensioned rockbolts are most effective in retaining loose blocks or wedges of rock near the surface of the excavation. These blocks may have been loosened by intersecting joints and bedding planes in the rock or they may have been created by poor quality blasting. In either case, falls of loose rock create unsafe working conditions and some form of support is required.

Since the amount of loosening does not usually penetrate very far into the rock mass, the support is only required to hold up the dead weight of the loose material. Mechanically anchored rockbolts, with the addition of mesh where small pieces of rock are likely to fall out between bolt heads, provide very effective support for these conditions. Tensioning of the bolts, usually to about 70% of their ultimate breaking load, is required in order to tighten the loose blocks and wedges and to provide as much interlocking between these blocks as possible. It is by helping the rock to support itself and by the prevention of further unravelling and deterioration of the rock mass that the tensioned rockbolts provide effective support.

Unfortunately, mechanically anchored rockbolts do throw several problems. There is a tendency for anchors to slip progressively with time, probably as a result of vibrations induced by nearby blasting. Hence, old rockbolts which have clearly lost all their tension are frequently seen in underground mines. Another problem relates to rusting of the bolts in rock masses due to groundwater, for example, in massive sulphides. Sometimes, the life of an unprotected bolt may be less than one year under such circumstances and, where long term life is required, the bolts should be grouted in place.

The need for mechanically anchored rockbolts is reduced significantly by careful blasting and by correct scaling. These techniques reduce the amount of loose rock which has to be supported and hence the need for bolts and mesh.

Grouted or Friction Anchored Dowels

One of the main disadvantages of mechanically anchored rockbolts is that, if the anchor slips or the bolt breaks, the capacity of the bolt drops to zero and the rock being supported can fall. This problem is less severe in the case of a fully grouted or friction anchored dowel because, even if slip does occur or if the face plate breaks off, the remaining length of the dowel is still anchored and will continue to provide support.

The problem with grouted or friction anchored dowels is that they cannot be tensioned and hence they have to be installed before significant movement has taken place in the rock. In fact, experience has shown that this apparent problem can be turned to advantage and that a combination of careful blasting and the installation of dowels as close as possible to the advancing face can provide very effective support for a much wider range of rock conditions than can be handled by mechanically anchored bolts. The installation of the dowels close to the advancing face ensures that relatively little movement has taken place in the rock mass and that the maximum amount of interlocking between individual pieces is retained. Retention of this interlocking is critical to the self-supporting characteristics of the rock mass and any loss of interlock causes a very severe drop in strength.

7.14.2 Shotcrete

Shotcrete is used very widely in civil engineering construction but is not used by the Mining Industry to the extent that it deserves. This is partly due to the fact that a typical mine has many working faces and it is difficult to schedule the shotcreting equipment efficiently. It is also due to traditional attitudes which are gradually changing in recognition of the fact the each element in underground support plays a different role and that shotcrete can be a very effective support medium. The shotcrete acts in much the same way as mesh that it prevents small pieces of rock from unravelling from the surface of an excavation. This helps to retain the interlocking and self-supporting characteristics of the rock mass. Since shotcrete is generally stronger than mesh, particularly if it is fibre-reinforced, and since it is corrosion resistant, it is generally considered to be a more effective support system. It is particularly useful in excavations, such as, ramps and haulages where long-term stability is important.

Shotcrete has developed into a versatile support system with the addition of microsilica and steel fibre reinforcement to the mortar/aggregate mix. The complex installation of thin layers of shotcrete, reinforced with weldmesh fabric can now be replaced quickly and economically by a single pass of steel fibre reinforced microsilica shotcrete. Sufficient research has now gone into shotcrete mix design and the constituent materials used, that shotcrete quality now rests almost entirely with the equipment operators. Shotcrete application requires constant attention to the supply pressure and volume of water, mix and air to ensure that the material leaves the nozzle in a continuous uninterrupted stream which can be applied by the nozzleman in such a fashion as to maximise compaction and quality while minimising rebound and overspray.

As shotcrete develops strength with time after application, it may be used effectively soon after excavation. Local readjustment of the in situ stress field due to mining is unlikely to induce excessive loading on the green shotcrete, and shotcrete has shown good resilience and durability to nearby blasting. As loading is transferred on to the support system, it gains strength and produces a stiffening support member. Most products are shot with up to 5% accelerator if a high early strength is required. This obviously leads to the development of a faster supporting member, but care must be exercised in design to ensure that the support will not become overstressed by load transferred from relaxing ground in high stress environments.

The use of a micro-silica additive means that rebound is reduced considerably, thickness of application can be increased, weak zones of rock with running water can be covered and voids can be filled effectively. Silica does not appear to affect the long term strength of the product.

The addition of high aspect ratio, deformed steel fibres, usually 30-38 mm long and 0.5 mm in equivalent diameter, enhances the post crack load bearing capacity of the support system, although it does not give a marked improvement to the initial bending strength of the shotcrete layer. Early problems with balling of steel fibres and excessive equipment wear have been largely overcome and the addition of steel fibres do not usually give rise to significant operational problem.

Typical Shotcrete Mix Design

Components	kg/cu.m	Percent
Cement	420	18.6
Micro-silica additive	42	1.9
Blended aggregate	1735	76.9
Steel fibres	59	2.9

7.14.3 Steel Sets

Steel sets have generally replaced timber as the traditional 'passive' support system in underground construction. The term 'passive' derives from the fact that the steel sets (or timber) do not interact with the rock in the same way as rockbolts or dowels. These elements become part of the rock mass in much the same way as reinforcing becomes part of the concrete in reinforced concrete. On the other hand, passive support, such as, steel sets can only respond to loads imposed on them by the inward movement of the rock. Since they are generally placed some distance behind the advancing face, most of the short-term movement in the rock has already taken place before the sets are in place and the only load which they are called upon to carry is the dead-weight of rock failing around the opening.

7.14.4 Friction Anchor or Split Set

Background

Developed by Scott in conjunction with the Ingersoll-Rand Company in the USA, this device has gained considerable popularity in the Mining Industry. As the split tube is forced into a drill hole, the spring action of the compressed tube applies a radial force against the rock and generates a frictional resistance to sliding of the rock on the steel. This frictional resistance increases as the outer surface of the tube rusts. The Friction anchor or Split Set is shown in Figure 7.34 (see Plate 21).

Advantages

It is simple and quick to install and is cheaper than a grouted dowel of similar capacity. It is found useful in moving and bursting ground.

Disadvantages

Cannot be tensioned and hence is activated by movement in the rock in the same way as a grouted dowel. Its support action is similar to that of an untensioned dowel and hence it must be installed very close to the face. The drillhole diameter is critical and most failures during installation occur because the hole is either too small or too large. In some applications, rusting has occurred very rapidly and has proved to be a problem where long term support is required. The device cannot be grouted.

Applications

It is used for relatively light support duties in the Mining Industry, particularly where short-term support is required. There is little application in civil engineering at present.

Typical Data

Yield load	130 kN (13 ton f)
Tube diameter	26 mm (1")
Hole diameter	33 mm ($1^3/_8$") to 39 mm (1 ½")
Lengths	Up to 8 m (24 ft)
Inflation pressure	20 MPa (3000 psi)

7.14.5 Grouted Cable Bolt

Background

Grouted cables were introduced to mining for reinforcement of the backs of cut and fill stopes. Cable reinforcement, using tensioned or untensioned, fully grouted cables, is very widely used in mining applications. Cables can be installed effectively in very narrow tunnels, they are inexpensive and have a very high load bearing capacity. The Grouted Cable Bolt is depicted in Figure 7.35 (see Plate 21).

Advantages

This system is inexpensive. If properly installed, it provides competent and durable reinforcement. It can be installed to any length in narrow areas. The system gives very high bolt loads in various rock conditions, as well as high corrosion resistance in permanent installations.

Disadvantages

Tensioning of the cable bolt is possible only if a special installation procedure is adopted. The use of standard cement in the grout requires several days curing before the cable can be loaded.

Applications

The system is increasingly used in mining applications.

Typical Data

Yield stress	1770 Mpa (257000 psi)
Yield load	500 kN (50 ton f)
Cable diameter	20 mm ($^{25}/_{32}$ ")
Hole diameter	35 mm ($1^3/_8$ ")
Lengths	Any length required

● ● ●

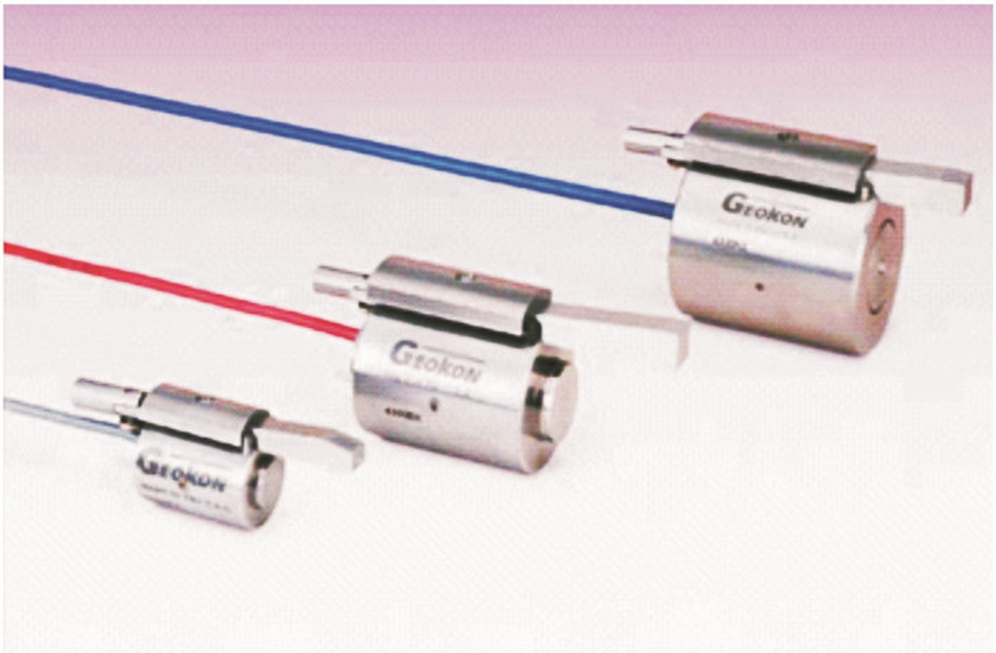


Figure 7.32 – Vibrating Wire Stress Meter Capsule

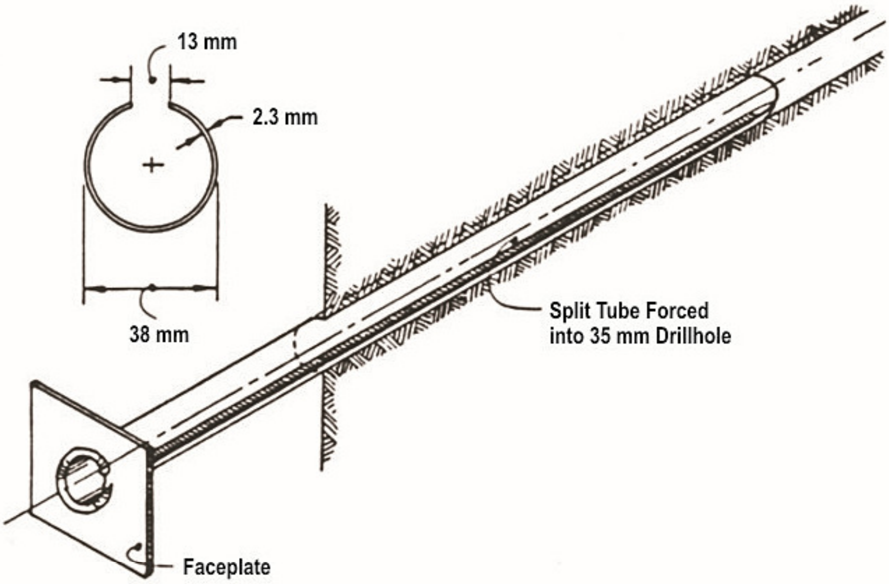


Figure 7.34 - Friction Anchor or Split Set

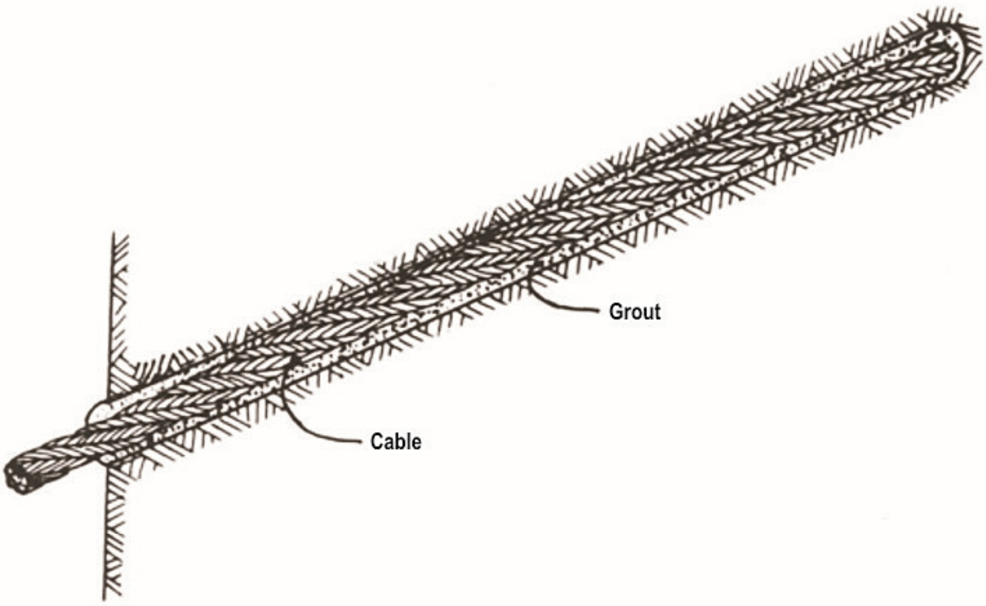


Figure 7.35 - Grouted Cable Bolt

Chapter

8 Application of Rock Mechanics in Opencast & Underground Mines — Case Studies

CASE STUDY - 1

8.0 SLOPE STABILITY INVESTIGATIONS FOR CALCITE AND WOLLASTONITE OPENCAST MINE

The principals of the rock mechanics are widely used for designing optimum pit slope of an open cast mine. Indian Bureau of Mines has conducted the study on optimum pit slope at wollastonite and calcite mine as a case study. The mine is in operation since last 30 years in environmentally fragile area. The mine produces about 55 thousand tonnes wollastonite per year and 60 thousand tonnes calcite per year. The recoverable reserve was about 0.83 million tonnes of wollastonite and 0.44 million tonnes of calcite at the present slope angle of 30°–33°. With the increase of production rate and high sticking ratio (1:3 to 1:6), it was felt necessary to maintain steeper slopes up to 45° to 50° in face number 1 and 7 so that 1.0 million tonnes of wollastonite and 0.45 million tonnes of calcite can be recovered. With the present geometry, about 83% can be recovered while the rest would get locked in the benches.

For the purpose of geo-technical investigation, the following items of work were carried out:

a) Geo-technical mapping of existing mine faces and the strata around the workings.

b) Geo-technical logging of available drill cores for RQD measurements

c) Collection and testing of rock sample for determining physio-mechanical properties from drill hole cores and mine faces

d) Processing and analysis of field data involving stereonet analysis, complications of RMR, Q and SMR classification. Computer analysis for slope stability and determinig the optimum pit slope angle

e) Redesigning pit slopes for ultimate pit limits

Based on mapping of the exposed cases of the benches, followed by litholoIgical logging of drill core, the following major rock types were exposed from hanging wall to foot wall:

a) Soil and Alluvium

b) Weathered Pegmatite/Pegmatite/Quartz vein

c) Limestone/Decomposed Limestone

d) Skam including bodies of Wollastonite and calcite

e) Granite

Detailed geo-technical mapping was carried out by mapping the exposed discontinuities (joints, bedding planes, shear zone etc.) on the scale of 1:200 and 1:1000. All the joints mapped were classified with reference to standard geomechanical system, namely, NGI (Q- System), RMR (CSIR) classification and SMR (Slope Mass Rating System).

For the purpose of collection of samples, appropriate sites were selected on the working benches and future mineable benches for assessing intact rock strength of different rocks. Overall rock mass quality of the existing formation for the underground mine development has been assessed. The RQD percentage was varied from 21% to 54% in both the deposits.

For the systematic application of rock mass classification approach the engineering properties, such as, uniaxial compressive strength, tensile strength, point load index and triaxial strength test were conducted on samples drawn from differerent litho units.

Based on the laboratory testing, the determined uniaxial compressive strength of pegmatite of Deposit No.1 was about 364 MPa, pegmatite of Deposit No.2 was 820 MPa, calcite 260-320 MPa and skam was 88 to 292 MPa . From the triaxial test conducted it was found that the skam samples showed cohesion from 7.75 to 11.00 and angle of internal friction was from 37° to 42°. Similarly, for wollastonite, cohesion was 13.66 MPa and angle of internal friction was 23°; for pegmatite cohesion was 6-9 MPa and angle of internal friction 37-40°; and for wollastonite, cohesion was 7.00 MPa and angle of internal friction was 30°. Based on laboratory testing results, the rock is strong and compact and can withstand eye confining pressure. The density test by volumetric method conducted showed that the density of skam was about av.3, pegmatite av. 2.5 and wollastonite av. 2.4. The stereonet analysis provides a good means of presentation of structural data generated by field mapping and the evaluation of rock mass characterisation with respective stability. The data on joint orientations and angle of internal friction are used as an input for performing the analysis. For the rock mass quality, the Q classification has been adopted. The average Q value for deposit No.1 was 7.46 and deposit No.2 was 8.2 . The rock quality was therefore assessed to be of fair quality. Slope mass rating classification has been adopted and rating found for Skam was 58, wollastonite 58 and pegmatite 57. The values, therefore, indicate that rock falls under fair category. Computer analysis for the plane failure and wedge failure analysis was conducted and it was found that there is no possibility of plane failure and wedge failure. The stereonet analysis reveal that it may be possible to maintain steeper pit slope angle up to 45° to 50° in footwall and hanging wall for both deposits. The present slope angle of 28° for deposit No.1 can further be steepened by back benching. Intermediate benches to recover wollastonite and calcite left within the benches were formed as it would be difficult to work outside the lease boundry. Due to proximity of seasonal stream, benches were formed in the footwall keeping safety berm of 15 m.

Geotechnical investigation has been conducted with a view to steepen the existing slope in order to recover the scarce material. Numerical modelling has been carried out using GALENA software for generating stability plots followed by sensitivity analysis.

Geotechnical instrumentation and analysis of monitoring data formed an integral part for obtaining necessary permission from Director General of Mines Safety (DGMS), during the mine planning and phasing of slope steepening operation. Slope monitoring and development of slope stabilisation methods formed an integral part of ongoing activity during the execution phase. The geotechnical instrumentation plan includes the following activities.

a) Pit slope monitoring

b) Phasing of excavation sequences

c) Providing supports at weak zones

During the proposed slope steepening operation, continuous surveillance of the slope has been proposed by survey stations fixed in the benches at specific locations for monitoring ground movements if any. Multipoint extensometer, invar-wire extensometer were installed at selected location to monitor the strata behaviour. Vibrating wire piezometer were installed to monitor ground water level within the pit bottom. Based on the monitoring data, slope reinforcement/stabilisation plan was drawn.

The specific support plan using (i) systematic bolting/cable bolting with wire mesh (ii) construction of pack wall/ buttresses/retaining wall was drawn.

As a result the steepening, ultimate pit slope was to be undertaken in two phases. Initially up to 45^{o} slope with slope reinforcement and slope monitoring and thereafter up to 50^{o} slope in the second phase depending on the strata condition. During the progress of steepening, it was proposed that necessary precaution be taken for rainwater run off and pit water through proper drainage so that monitoring help in carrying out slope stability steepening could be taken effectively.

8.1 GEO-TECHNICAL INVESTIGATIONS FOR ASSESSMENT OF STABILITY OF TAILINGS DAM AT GOA

Geo-technical investigations were carried out for the assessment of stability of Tailing Dam. The iron ore tailings are finely ground material. There were three tailing ponds . The tailing ponds are desilted after drying of tailings and dry tailings are stacked separately. The present capacity of silt in tailing pond No.1 was 6 lakh cu.m., tailing pond No.3 was 4 lakh cu.m. and tailing pond No.4 was 12 lakh cu.m.The ground water was encountered at borehole near Vagus colony from 32 meter with a yield of 100 gallon per hour; at 40.2 m, the yield was 1000 gallon per hour; and at 41.8 m, the yield was 2000 gallon per hour. The yield of ground water of 90 gallon per hour was encountered in the borehole drilled near Pilot plant. Thus water level was found to lie in between 25 m to 40 m. Tailing ponds were systematically sampled using pipe sampling method and grab sampling. The purpose of this method is to draw in situ sample for laboratory testing. Based on the grained size analysis, it was observed that at > 7% fraction, the weight percent is 33 % which indicates that the sample contains about 33% clay and 67% silt. The sample, therefore, can be termed as silty clay. According to direct shear test the cohesion was 4.4 tonnes per sq.m and angle of internal friction was 30–32^{o}. As per testing the dry density was 2.15 to 2.64 gram per cu. m and the sp. gravity was 3.1 to 3.4. The permeability test was conducted and coefficient of permeability was found to be about 6.0×10^{-6} cm per second.

Tailing Dam can be constructed using upstream method, centerline method and downstream construction. The beach slope which usually varies with core type physical properties of tailings, deposition, flow rate method, shape and size. The average rate of dry density falls in the range of 1.0 to 2.0 tonnes per cu.m. considering mineralogy and settling properties of tailings. The maximum allowable rate of rise is, therefore, largely dependent on drying conditions and consolidation characteristics. The maximum rate of rise should be in between 1.5 and 2.5 m. The overall embankment slope extends from downstream edge of the crest at the top height of the embankment. The average slope is less than the stable slope of any intermediate embankments and is largely dependent on overall stability. The overall stability depends upon the shear strength of deposited tailings near embankments (with potential failure zones) and has bearing on consolidation characteristics. The embankment geometry is dependent on numerous factors including stability and safety, access, availability of construction materials and environmental aspects. The long term stability of overall slope is generally analysed using total stress analysis. The main factors affecting dam stability are embankment height and slope, the nature, strength & degree of compaction of foundation and embankment materials. The stability of embankment can be evaluated by using in situ shear strength results. Effective seepage control measures can be achieved through use of appropriate embankment construction materials having low permeability, installation of seepage cut off or seepage trenches, minimising water ponding on storage facility and maximising the extraction of water from tailings through thickening. The basic data required for the stability assessment are field test & laboratory data and site conditions. The analysis can be carried out using limiting equilibrium method. Bishop circular failure method is best suited for such type of analysis. Bishop circular method of analysis can be performed to understand the effect of the factor of safety with reference to rising the height of the dam in stages by maintaining a free board of 1.5 m. Ground water pressure (piezometric) is the most important parameter affecting the stability of the structure which need to be defined realistically for different problems. The pheratic surface is based on the evaluation of the rainfall data followed by the data on ground water table observed from boreholes.

8.1.1 Bishop's Method

Bishop method assumes a circular failure surface. Bishop's simplified method of slices is used for analysis of circular in slopes cut into materials in which failure is defined by the Mohr-coulomb failure criterion. Bishop method include tension crack on the surface, pheratic surface, failure through toe of slope, density of water, density of rock etc. With wide availability of computer programme, the iterative procedure required for obtaining a factor of safety is not tedious. The failure surface assumed for the first analysis may not give the lowest factor of safety and one should repeat the entire calculation a number of times in order to find the failure surface with the lowest factor of safety. Stability of tailing dam investigated by Bishop method is shown in Figure 8.1.

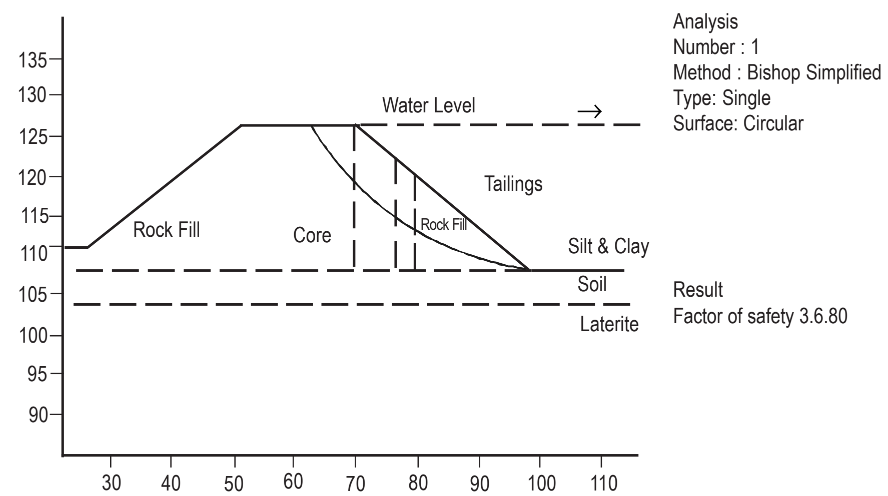


***Fig. 8.1**: Stability of Tailing Dam by Bishop Method*

The stability analysis has been carried out using GALENA software based on the limiting equilibrium method.The factor of safety has been computed by two different methods. In the first method both piezometric and pheratic surface were considered during modelling. In the second method only pheratic surface has been considered during modelling. If the height is increased up to 5 m using upstream method the factor of safety was worked out to be 2.08 which indicate likely stable conditions to be envisaged during rasing. It was observed that that there is no seepage of water from the dam in the downstream direction and even though the dam height is raised through stages at a rate of 1 to 1.5 m.

8.1.2 Geo-Technical Investigations at Underground Mine

The geo-technical investigation was carried out at copper, lead and zinc mine. The objectives behind the investigation was to assess the stability of strata conditions of 38.6 meter; to assess the thick barrier pillar laying below the river; and also to assess the strata condition between third and fourth level in which experimental cut and fill stoping method was proposed. The South-western portion of the orebody situated below the river is rich in copper mineralisation, therefore to safeguard the workings below the river the thick barrier pillar needed to be assessed.

Cut and fill stoping is a regular cycle of operation, which involves drilling, blasting, mucking and concurrent filling. In order to maintain uniform mill feed grade and material to back fill the worked out stopes, atleast two or three stopes block should be ready to meet the daily production. Therefore, geotechnical investigation was carried out to assess the envisaged strata condition during the cut and fill stoping method proposed in between third and fourth level, by opening of atrial stope block and to monitor the strata conditions during the progress of stoping.

Geotechnical mapping of barrier pillars drive of 1.8 m x 0.8 m having strike length of 180 m was carried out. Geo-technical instrumentation was carried out by installing multi-point borehole extensometer at barrier pillar drive. Instrumentation of experimental cut and fill stope area was carried out by installing borehole extensometer/stope convergence instruments using tape extensometer and installation of two numbers load cells (20 to 50 tonnes capacity) to measure the incoming load on the supports provided in cut and fill stoping and in the barrier pillar drive.

The ore lenses was bounded by quartz-sericite schist, chlorite schists on the hanging wall as well as in the foot wall. Width of the ore body that varied from 1 m to 11 m was found at the southern portion of the ore body. The ore contact with the wall rock are sheared. The host rock of mineralisation "epidiorite" is usually hard and compact. The foot wall rocks are also relatively hard and compact as compared to hanging wall formations which are weak and friable. The structure copper, lead-zinc ore body is complicated by faults, joints, shears, slickensides and foliation.

The detailed geo-technical mapping of 1.8 m x 1.8 m drive having length of about 1.66 m was carried out. The overall rock mass rating of the barrier pillar falls under fair category. Average RMR value is 54.16 in CSIR RMR geo-mechanic classification. In NGI classification, the rock mass falls under the good category having an average Q value of 4.26. Geo-technical mapping followed by visual inspection shows that there is no indication of spalling, slabbing or rock mass distress.

The stereonet analysis of joint orientations observed in the barrier pillar drive and in the vicinity of stopes indicated the presence of three prominant joint sets with limited continuity. The joints are slightly altered, rough and slikensided. The population matrix of the joint set can be grouped into:

Set No.1 (J1) : N10°W-S10°E/50°W

Set No.2 (J2) : N89°W-S89°E/60°S

Set No.3 (J3) : N50°E-S50°W/55°SE

The principal stress direction derived from stereonet analysis has revealed that the orientation of major principal stress is at S40°W, intermediate principal stress direction is at N50°W and the least principal stress direction is at N40°E. The strike direction of the present workings are aligned more or less along the least principal stress direction.

The in situ stress field computed for different working depths of 38.6 m (Barrier Pillar Drive), 63.0 m (proposed Cut and Fill stope) and 87 m(Present depth of workings up to 5th level) has indicated the horizontal to vertical stress ratio (h/v) that varied from 2.93 to 2.86 .

The calculated horizontal stress value was 2.2 times the vertical stress. This relationship holds good and was established worldwide by in situ stress measurements carried out in different mines.

The vertical stress computed for a depth of 38.6 m has recorded a value of 0.96 Mpa, similarly,for a depth of 63 m the value indicated was 1.57 Mpa and to a further depth of 87 m the value indicated was 2.17 MPa. Thus the vertical stress gradient increased by 1.0 MPa inferring that there is no significant effect of vertical stress on mine openings.

The intact rock strength of hangwall, ore zone, footwall formations resulted in an average Uniaxial compressive strength of 90 MPa which is classified as "medium rock" with an estimated cohesion value of 0.1 to 0.2 MPa and angle of internal friction that varied from 30^{o} to 40^{o}.

The overall rock mass rating (RMR) obtained for orebody is 56.0, for hanging wall 53.0 and for footwall formations is 58.0. In the NGI classification, the 'Q' values for orebody is 5.32, hanging wall 5.30 and for the footwall 5.33 respectively.

From the analysis of overall rock mass rating, the estimated span of unsupported excavation works out to be 8 m to 10 m. When stoping progresses, depending on the site conditions, longer spans can be considered with proper ground support. Based on the existing knowledge of the strata conditions, a plan of underground instrumentation was prepared and implemented at the mine. Nine boreholes extensometer were installed in the pillar drive and other selected locations to monitor the ground movements (Fig. 8.2). In addition, tape extensometers, load cells (20 T capacity) and LVDTS to moniter stope convergence, particularly, load behaviour on supports during Cut and Fill stoping were also installed. Analysis of data from ground monitoring instrumentation did not reveal occurrence of any significant movement during the progress of stoping.

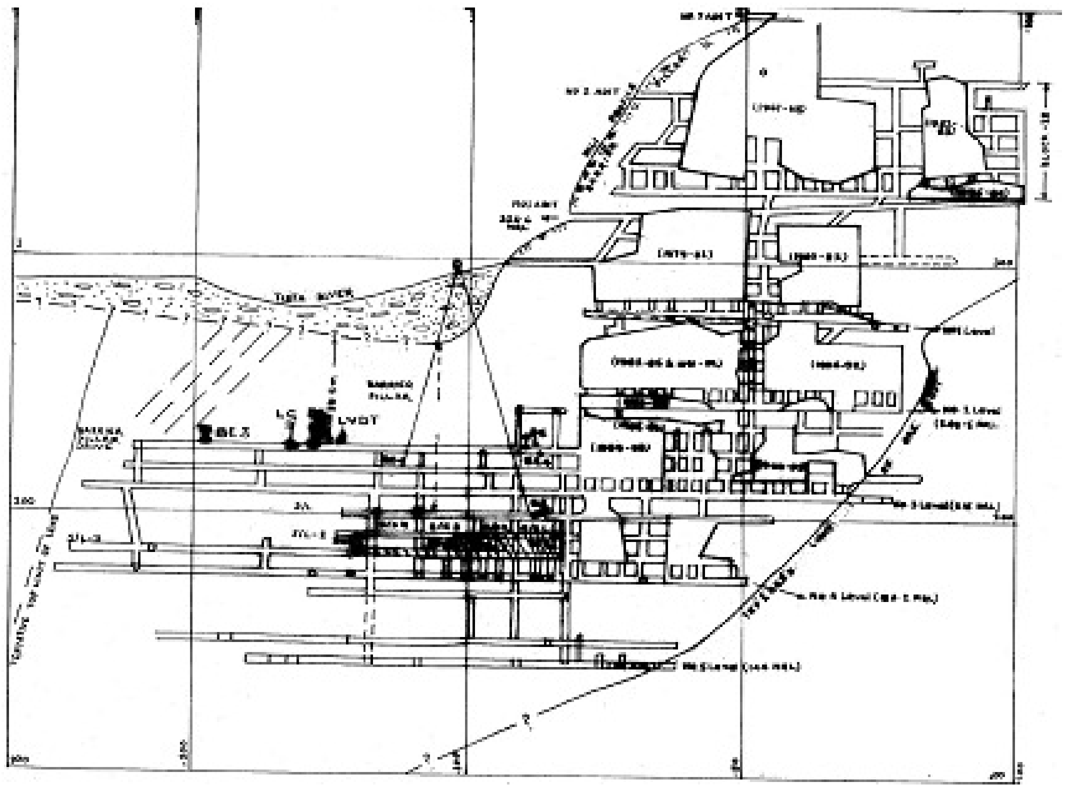


Figure 8.2: *Longitudinal Vertical Section of Underground Mine Showing Summary of Rock Mechanics Instrumentation*

Figures 8.3 & 8.4 depict underground monitoring (see Plate 22).

The condition of water percolation/ sleepage in the underground workings were monitored closely at regular intervals. The analysis of data obtained so far indicated that the workings fall under 'Dry and Moist' category. There was no significant change in the ground water seepage conditions.

The support analysis on the rockmass, fill material and on timber supports of the proposed Cut and Fill stope indicated that an unsupported span of around 8–10 m can be kept during stoping. However, the maximum stand up time of unsupported excavation was worked out to be around 2 months. The analysis also showed that systematic and concurrent filling of stope would minimise the stope wall convergence.

For the purpose of monitoring, Load Cells (20 T to 50 T capacity) were installed at the stope back over timber support to measure load deformation. Physiomechanical properties of the fill material and analysis of support characteristics of fill material are vital parameters required for designing proper support system. The envisaged stope recoveries from the proposed cut and fill stoping would be 50%.

8.1.3 Geo-technical Investigations at Perlite Mine

Indian Bureau of Mines has carried out investigation for the purpose of (i) collection and testing of rock samples from the mine adits for determining the physio-mechanical properties of rock and rock mass classification (ii) carrying out numerical modelling studies of stope area for determining the maximum width of the galleries, height and size of pillar, factor of safety and stand-up time (iii) conducting blast vibration study to assess the likely effect of blasting in the neighbouring areas as per technical Circular No.7 of 1997 issued by DGMS.

The mine is located on an isolated hill amidst rock type, i.e., basalt, rhyolite, perlite and limestone and nearby a temple.

The geotechnical investigation was carried out for adit No.1 and adit No.2. The rock conditions exposed in the adit and cross cuts were mapped in detail by scanline mapping. About 14 samples were collected from cross cuts and roof of adit for assessing the physio-mechanical properties of rock. Since no borehole was drilled inside the adits or in the neighbourhood areas, it was not possible to assess various strength properties of adjacent rock. Therefore, representative block samples were collected from different locations of the adits.

Adit No.1 was advanced up to 46 m with maximum 3.4 m width and Adit No.2 was advanced up to 44 m with 2.4 m maximum width. The height of the adits varied from 2 m to 3.3 m. Both the adits were parallel and connected with cross cuts. There was seepage of water in the adit. The roof and sites of adits were strong and therefore no support was required during the development. The samples of rhyolite, perlite and basalt collected from the adits were tested in the laboratory and the test results are as below (Refer Tables 8.1 & 8.2).

Table 8.1: Summary of Test Results

S. No.	Rock Type	No. of Samples	Size (cm)	Average Load at Failure (kN)	Uniaxial Compressive Strength (kg/cm^2)	Remarks As per ISRM
1.	Basalt	5	8.0 x 8.0	800	1250.00	Very strong rock
2.	Rhyolite	4	8.0 x 8.0	300	468.75	Medium strong rock
3.	Perlite	5	8.0 x 8.0	450	703.13	Strong rock

The uniaxial compressive strength of basalt, rhyolite and perlite has shown the rock as very hard and strong and that which can withstand high confining pressure. Based on geological mapping, the values of 'Q' and 'RMR' for each scanline have been determined and these are depicted in Table 8.2.

Table 8.2 Rock Mass Classification of Adit No.1 and 2

Scanline No.	Av.RQD	Jn	Jr	Ja	Jw	SRF	Q	RMR
1.	62.50	5	1.5	0.75	1.0	2.5	10.00	53.00
2.	56.66	4	1.5	0.75	1.0	2.5	11.33	53.48
3.	70.00	5	1.5	0.75	1.0	2.5	11.20	53.44
4.	78.75	3	1.5	0.75	1.0	2.5	21.00	55.89
5.	70.00	3	1.5	0.75	1.0	2.5	18.66	55.43
6.	59.00	9	1.5	0.75	1.0	2.5	5.25	50.48
7.	74.54	3	1.5	0.75	1.0	2.5	19.87	55.68
8.	80.00	3	1.5	0.75	1.0	2.5	21.33	55.96

The geomechanic classification of the rocks places them under the 'fair category'. The Q values varied from 10.00 to 21.33 and RMR values varied from 50.48 to 55.96, which indicate that the rock mass is stable and did not require any support. Design parameters for optimising pillar height, width and gallery sizes were formalised.

The load to be borne by pillar would depend on the strength of the floor and roof material. The optimum width to height (w:h) ratio of 10:1 is required for the purpose stability of pillars. The pillar strength also depends on the rock mass strength of the pillar material, shape & size of the pillar and discontinuities therein. Empirical procedure was adopted for calculation of factor of safety (FS). For pillar of width : W_p= 4 m; height : h = 6 m; width of the opening W_o= 12 m and 14 m, depth from the surface = 50 m; and unit weight of rock mass γ = 0.2 MN/m^3; with Q value of 21.3; and RMR value of 55.9; and MRMR value of 52.0; and that whose rock mass falls under 'Fair category', the value of the pillar strength/uniaxial compressive strength, i.e., $\sigma s.av/\sigma c$ for W_p/W_h = 4 m/6 m = 0.66 and for fair quality rock mass, the value of average pillar strength/UCS which is $\sigma_{ls.av}/\sigma c = 0.55$ (where, σc = 70.3 MPa for perlite) and $\sigma_{ls.av}$ = 0.55* 70.3 = 38.6 MPa and the average pillar stress σ_p for rib pillar is $\sigma_p = \gamma .z\ (1+ W_o/W_p)$. σ_p = 0.2*50 (1+12/4) = 30 MPa, the factor of safety of the pillar is given by the formula,

FS = Av. Pillar strength/ Av. pillar stress, i.e., FS = 38.6/30 = 1.3.

Similarly, for the width of opening W_o = 14 m, considering W_p/W_h = 0.66, the factor of safety works to be 0.9, for the perlite bearing pillars.

The stability of the pillar usually depends upon size, shape, elastic properties, rock mass quality and amount of deformation. It would also depend on the roof and floor rocks and ratio of span of yielding zone to depth below the surface. The size of the pillar should be kept wider to avoid failure. The shape of the pillar was kept with W:H ratio, i.e., 4 m:6 m. In future, if the pillar width needs to be extended at the base, the (W/H) ratio of 4:1 would be an ideal proposition. The pillar has been designed for the factor of safety of 1.5 for the ultimate extraction of pillars.

For W:H ratio less than 4.5 :1

Pillar strength Ps $= 133\ W^{0.50}/H^{0.75}$

$= 133 \times 4.0^{0.50}/6.0^{0.75}$

$= 59.1$ MPa

From the empirical relationships, it has been revealed that there is no significant effect of normal stress, and unsupported stope span of 12 m can be achieved, for pillar width of 4 m. The factor of safety for the rib pillars works out to be 1.3, with pillar size of 4 m * 6 m (W/H), with stope span of 12 m.

Numerical modelling using 'N FOLD' programme has also indicated that the strata conditions are stable with 4 m* 6 m (W/H) of pillar dimensions, with 10 m, 12 m, 14 m spans of stope. The analysis has also indicated that there is no influence of horizontal or vertical stress regime, and hence stope spans and pillar widths can further be optimised during stoping operations, keeping safety considerations.

The optimum height of the gallery depends upon the thickness of the perlite band to be mined. In this case, the average thickness of perlite band is 5 m. Therefore, for the maximum height of the gallery of 6 m, the width of rib pillar works out to 4 m. With this approach, the entire thickness of the perlite band can be mined out.

8.1.4 Blast Vibration Study

Blast vibration study was carried out in order to investigate effect of the blasting on the temple located near the mine. Portable field seismograph "INSTANTEL" used for recording the data and same was analysed by using software supplied with seismograph. The ground induced vibrations caused due to underground mining activity was monitored by recording the vibration levels at different locations on the surface. Based on the recorded data of the seismograph it was revealed that the vibration levels were within the permissible limit as described by DGMS.

The effect of the ground vibrations due to blasting can further be minimised by choosing proper explosives, including the existing blasting practices by introducing perimeter blasting, pre-spliting etc.

8.1.5 Design of Stoping

The perlite band is flat to gently dipping having varying thickness from 3 to 8 m. The roof and the side walls are competent having 60–80% RQD and hence there is no support required.

To evaluate the ground condition, the situation was simulated in N-Fold Software. It is a pseudo three-dimentional stress/ displacement analysis programme based on displacement-discontinuity method. The method treats the stope as thin opening. The periphery of the opening needs to be discretised (Being Boundary Element Method). The programme calculates stresses and displacements at the periphery of the opening.

Input Parameters

To model, the mining excavations, the code needs input of (a) geometry (b) in situ stress regime (c) physical properties of the rock mass (Youngs modulus, Poisson's ratio etc.) and (d) width of the proposed excavation (Refer Figures 8.5 & 8.6) [see Plate 22].

In situ Stress: Since the excavations are very small and at shallow depths, the stresses are not of significance. Visual inspection of underground excavations has indicated no signs of roof fall or loose formation. Therefore, the in situ stress field has been estimated, based on the following equations:

$$\sigma_{Hor} / \sigma_{vert} = 3 - \text{Depth}/500 \ (\text{Depth} < 1000 \text{ m})$$

$$\sigma_{Hor} / \sigma_{vert} = 3 - 50/500 = 2.9 \text{ MPa}$$

$$\sigma_{vert} = 50/40 = 1.25 \text{ MPa}$$

From the above results of stress, it has been concluded that due to shallow excavation (max. 50 m from the surface of the hill) there will be no influence of horizontal and vertical stresses.

The input data utilised for running 'N-Fold' software include the following:

1. Uniaxial compressive strength σc = 46 MPa (average)
2. E rock mass = 20 GPa
3. λ = 0.2 MN/m^3
4. Material = Linear elastic
5. Thickness of ore body= 5 to 10 m (mineable)
6. Z = 50 m; Dip = 2^o to 15^o
7. Boundary condition = All sides clamped

   σ_1 = acting along strike

   σ_2 = acting parallel to strike

   σ_3 = 0.03 * H MPa (acting vertically down)

   where, H is depth of the working below the hill top (in meters)

Physical properties of the rock material :

Young's modulus of host rock = 20 GPa

Young's modulus of ore body = 20 GPa

Poissons's ratio : 0.2

Width of the ore body: 5 m (For modelling)

Width of the host rock: 10 m (For modelling)

Recommendations

1. The change per delay should not exceed 2 kg of explosive, subject to maximum of 20 kg of explosive per blast.
2. By the method designed, about 25% of the perlite should be left in situ as ribs/or as pillars to safeguard the stability of excavation/opening. The non-effective width will be around 10 m.
3. In case of development heading, the size of the excavation should not be more than 3 m (H) and 3.5 m (W).
4. The void created due to mining should be filled with waste rock by making pack walls.
5. In case of stopes, where the thickness of perlite band is more than 3.5 m, cut and fill method must be practised. So that at any point of time, the opening is not more than 3 m.

CASE STUDY - 2

8.2 EMPIRICAL SLOPE DESIGN FOR FRIABLE ORE BODIES — GOAN IRON ORE MINES

Slope stability analysis forms an integral part of any opencast mining operation. It has been observed that several opencast mines experience slope instability during the operational cycle due to several contributing factors. In some mines, local bench failures are common phenomena, which may further lead to catastrophic failure. The ideas outlined in literature concerning rock slope failure mechanisms and the appropriateness of different pit slope design are diverse. To overcome these uncertainties there is a need for development of new generation of slope design guidelines for mining weak and friable ore bodies with weak wall rocks to continue mining up to the planned ultimate pit limits. Most of the pit slope failures are associated with governing factors, such as, pit geometry, slope angles, bench height, rock mass strength, excavation sequence and water regime. Even though the mechanism of slope failures are known, the possibility of slope failures could not be predicted with reasonable degree of accuracy in weak rock material. The standard geomechanics classifications for the design of slopes are meant for hard rock mines, where discontinuities play a major role in causing structural instability. Whereas in the case of soft and weak rock masses, the cohesion and angle of friction play a major role in governing the stability of slopes. An attempt also has been made with empirical approach for developing a new generation of slope design guidelines based on Slope Rock Mass Rating (SRMR) classification.(Robertson 1988). Slope performance curves were developed in the form of nomograms for weak rock mass classification. The nomogram can be used as one of the design tools for establishing the correlations between slope height, slope angle and its corresponding factor of safety.

Slope control in intensely weathered and fractured rock masses is a matter of concern for the design of safe slope angles in the operating life of the mine. The strength properties of the soft rock/soil material and its classification in terms of stability analysis are considered to be difficult tasks. Difficulty is still encountered in rock mass strength determination in those instances where the rock material strength is very low and variable, intensely weathered, fractured, which do not fit into the standard Geo-mechanics classification systems. Frequent occurrence of slope failures were observed in several Goan mines usually after the onset of monsoon, due to high degree of saturation and low shear strength properties of excavated slopes. The variability in rock mass strength can be attributed to degree of saturation, the structural/deformation history of the area. A classification system for weak rocks was developed by Robertson et al. (1988) based on their shear-strength properties. Attempts also has been made to develop a nomogram based on SRMR classification, slope angle, slope height and its corresponding factor of safety. The mechanism of slope failures in weak rocks are very complex and dependent on failure pathways, in which certain rock units fail first followed by subsequent failures due to redistribution of stresses from the preceding zone. The results of field observations, laboratory testing of slope forming materials and slope monitoring have led to an awareness of various mechanisms of failure and the conditions under which they are likely to occur. Whereas in the real world situation, the slope failure mechanisms are much more complex involving many other variables due to complexity within the geological materials. The testing techniques for material properties enable weak zones to be identified and their relative strengths to be determined. A good understanding of the historical data on standing slopes and failed slopes and the material in which the slopes were excavated will provide a better understanding of the failure mechanism.

8.2.1 Slope Stability Problems in Weak Ore Bodies—Goa

The lithological formations of iron ore mines in Goa are known to comprise weathered laterite, weathered dykes, manganiferous clay, weathered phyllite and transitional ore, lumpy ore, alumina-rich iron ore fines, siliceous fines, which can be attributed as "soil type" material. In reality, these formations are not homogenous in nature both in terms of strength parameters and ground water circulation. Furthermore, the stability or instability of these formations is to a certain extent governed by relic discontinuities, such as, joints and faults present in the original formations, which are hard to detect.

In terms of soil mechanics, the analysis is based on the combination of saturated-unsaturated seepage analysis and circular arc stability analysis.This method can be used to evaluate the reduction of soil strength of the formations along the slip surface, the increase in the self weight of a slope due to seepage and the excess pore water pressure developed in the saturated zone. For performing stability analysis, data on material strength properties, permeability & porosity, water regime, pit geometry were considered. For the obtaining the boundary conditions for saturated-unsaturated seepage conditions, the shear strength, coefficient of the permeability of saturated and unsaturated formations were analysed. For different types of lithological formations, the slope stability analysis can be divided into two major classes—Gross stability and Local stability.

"Gross stability" refers to large volumes of material and gross instability occurs in case of deeply weathered rocks and soil type of materials, which may cause large "rotational type" shear failure (Refer Figures 8.7 & 8.8—Plate 22).

"Local stability" refers to much smaller volumes, the corresponding failure that can affect one or two benches at a time are due to:

- shear planes, jointing,
- slope erosion due to surface drainage, and
- "piping effect" caused by ground water flow emerging at the ground surface.

The problem of local instability can be serious and can sometimes lead to gross instability if not attended in time. The effect of surface erosion in certain open-pit slopes is particularly strong and damaging, which deserves much attention. At many such mines, little or no attention is given to surface run-off because the importance of a fast building of hydrostatic head on stability is not well understood. There are, however, a few mines that did comprehend the fact that catastrophic failure may occur, during heavy rains in a slope that is already near failure. The situation can be particularly worse if there are already existing tension cracks in the slope and no means of diverting the run off water away from the pit area have been made.

8.2.2 Mechanism of Slope Failure in Soil Type of Material

The causes which concur to slope failure are many. Among them, only one major cause that is gravity a constant force ensuring that anything loose move downwards is comprehensible. The principle of the mechanism of failure is simple—there are disturbing forces due to gravity (including water pressure) and resisting forces due to the strength of the soil and rock material that are at play. The factor of safety (F) is defined as the ratio of resisting forces (S) to disturbing forces (τ).

$$F = \frac{S}{\tau} \quad (1.1)$$

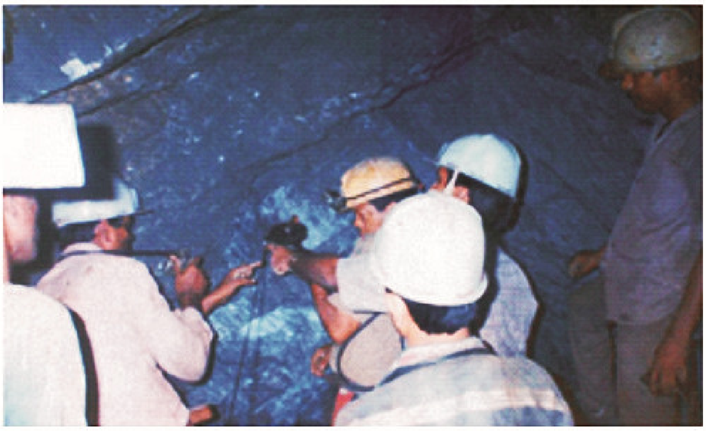


Figure 8.3 - Measurement of Convergence in Cut and Fill Stope

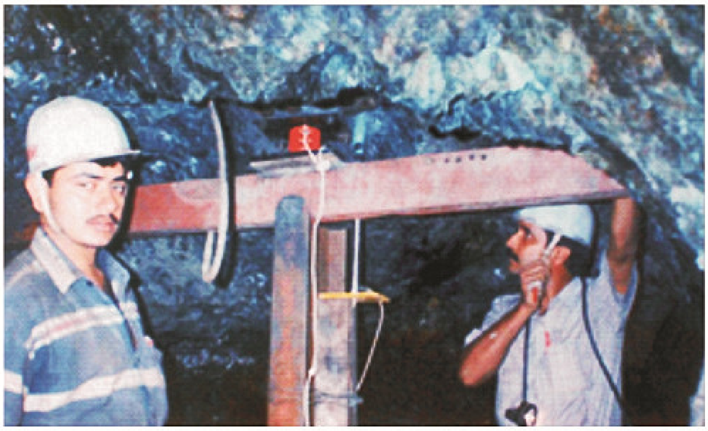


Figure 8.4 - Installation of Load Cell and LVDT in Barrier Pillar Drive

Figure 8.5 - Portal of Adit No. 1

Figure 8.6 - Underground Blast Monitoring on the Surface of Hill

Figure 8.7 - Gross Instability of Benches: Goa

Figure 8.8 - Local Bench Failure

If $S > \tau$, then F>l which connotes that the slope is stable. The factor of safety can drop below unity due to either reduction of resisting forces or due to increase of disturbing forces. To stabilise a slope, either the disturbing forces must be reduced or the resisting forces must be increased. The negative role of water pressure in the soil (u) can be enhanced by the presence of tension cracks behind the top of the slope. In case of heavy rains these cracks can become water-filled and the ensuing pressure results in a horizontal force (V), adding further instability to the slope. The condition in which the disturbing forces (τ, shear stress) equal the resisting forces (S, shear strength) is called a condition of "limiting equilibrium", at which the slope is on the point of failure:

when, F = l, ($S = \tau$) .. (1.2)

There are two primary ways in which the safety of a slope can be increased

1. By decreasing the shear stress in the soil (τ),
2. By increasing the shear strength in the soil (S).

A decrease of shear stress (τ) can be accomplished by the following:

- by decreasing the inclination of the slope above the potential failure surface
- by removing laterite cover/soil from the head of the slope
- by unloading the head of the slope and applying the same load to the toe of the slope—an increase in S can be achieved,
- by decreasing the water pressure in the soil, and
- by increasing the shear strength parameters of the soil c' and φ'. For practical purposes we can assume that c' and φ cannot be improved. To obtain maximum soil strength and maximum inclination of the excavation slope, it is therefore necessary to minimise the pore water pressure by draining the slope effectively.

By understanding the shear strength parameters, such as, sliding, angle of friction and cohesion, it is possible to reasonably gauge the pit slope behaviour due to various stresses, shear strength parameters. It is very important to understand the role of pore water pressure in slope. Therefore, by minimising the pore water pressure through proper drainage methods, the maximum soil strength can be achieved with maximum inclination of the excavation slopes.

In order to understand the mechanism of different types of slope failures, back analysis of historical data on failed and standing slopes has been performed (Figure 8.9—Plate 23).

From Figure 8.9 a series of points were plotted—identified by the geometric parameters, such as, height and inclination of a number of slope excavated in different materials.

Figure 8.10 (see Plate 23) was simplified since some variables influencing slope stability, such as, geological conditions, water pressure in the soil, etc. were not independently introduced. It has been observed that:

- geological conditions in the different mines are rather uniform,
- the clayey materials show similar behaviour, and

all slopes failures recorded followed the same time pattern. From the analysis of data, it has been observed, that slope failures have no direct bearing with the slope angle and slope height and the material in which slopes were excavated. The initiation of slope failure mechanism starts with development of tension cracks followed by widening of cracks over time on the slope crest. The laterite overburden material resting on the top of the benches also acts as dead weight, which also cause slope instability. Another major contributing factor is ingress of rain water into slope and consequent build-up of pore pressure in the highly permeable slope forming materials, such as, manganiferous clay, phyllitic clay, weathered and altered dykes and ore zone, could potentially trigger the failure at a limiting equilibrium condition. Figure 8.10 (see Plate 23) illustrates a typical case study on the effect of limiting equilibrium conditions of slope behaviour during monsoon season leading to its ultimate failure.

From Figure 8.10, it can be been observed that factor of safety is sensitive to limiting equilibrium conditions of slope. The slope could stand for 5 years as stable slope and after steepening the slope, the limiting equilibrium conditions were disturbed leading to ultimate slope failure.

8.2.3 Available Slope Performance Curves

Slope performance curves provide a valuable tool in the design process where rock mass failure plays a strong control in the stability of slopes. The curves are derived from the performance of stable and unstable slopes plotted on a slope angle versus slope height plot. The curves are often site specific and take into account the impact of existing failures, i.e., the remaining time frame for mining and the acceptable risks to the mining operation. This line can therefore be used as a guide for upper bound heights and slope angles that can be considered in slope design.

8.2.4 SRMR Classification System

The classification system for weak rocks developed by Robertson et al. 1988 is considered appropriate to classify the rocks in terms of its shear strength properties for stability analysis, i.e., RMR values less than 40. As this method did not allow for consistency in strength assessment (i.e., different rock mass rating methods above and below RMR = 40), Robertson (1988), proposed the SRK Geomechanics classification (SRMR) as shown in Table 8.3.

Table 8.3: SRK Geomechanics Classification: Slope Rock Mass Rating (SRMR)

PARAMETER		RANGE OF VALUES										
Strength of intact rock material	Is 50 (MPa)	> 10	10 – Apr.	4 – Feb.	2 – Jan.	For this low range UCS test is preferred						
	UCS	R5	R4	R3	R2	R1	R1	<1				
	(MPa)	>250	100–250	50–100	25–50	25–May	5–Jan.	S5 S4 S3 S2 S1				
Rating		30	27	22	19	17	15	10	0	2	1	0
Handled RQD (%)		90–100	75–90	50–75	25–50	<25						
Rating		20	17	13	8	3						
Handled (mm) discontinuity spacing		>2000	600–2000	200–600	60–200	<60						
Rating		20	15	10	8	5						
Condition of discontinuities		Rock >R1	Rock >R1	Rock >R1	Rock >R1	Rock <R1						
		Very rough surfaces	Slightly rough surfaces	Slightly rough surfaces	Slickensided surfaces	Soft gouge >5 mm thick						
		Not continuous	Separation < 1 mm	Separation < 1 mm	OR Gouge < 5 mm OR	OR						
		No Separation	Slightly weathered walls	Highly weathered walls	Separation 1–5 mm continuous	Separation >5 mm						
		Unweathered rock walls				Continuous						
Rating		30	25	20	10	0						

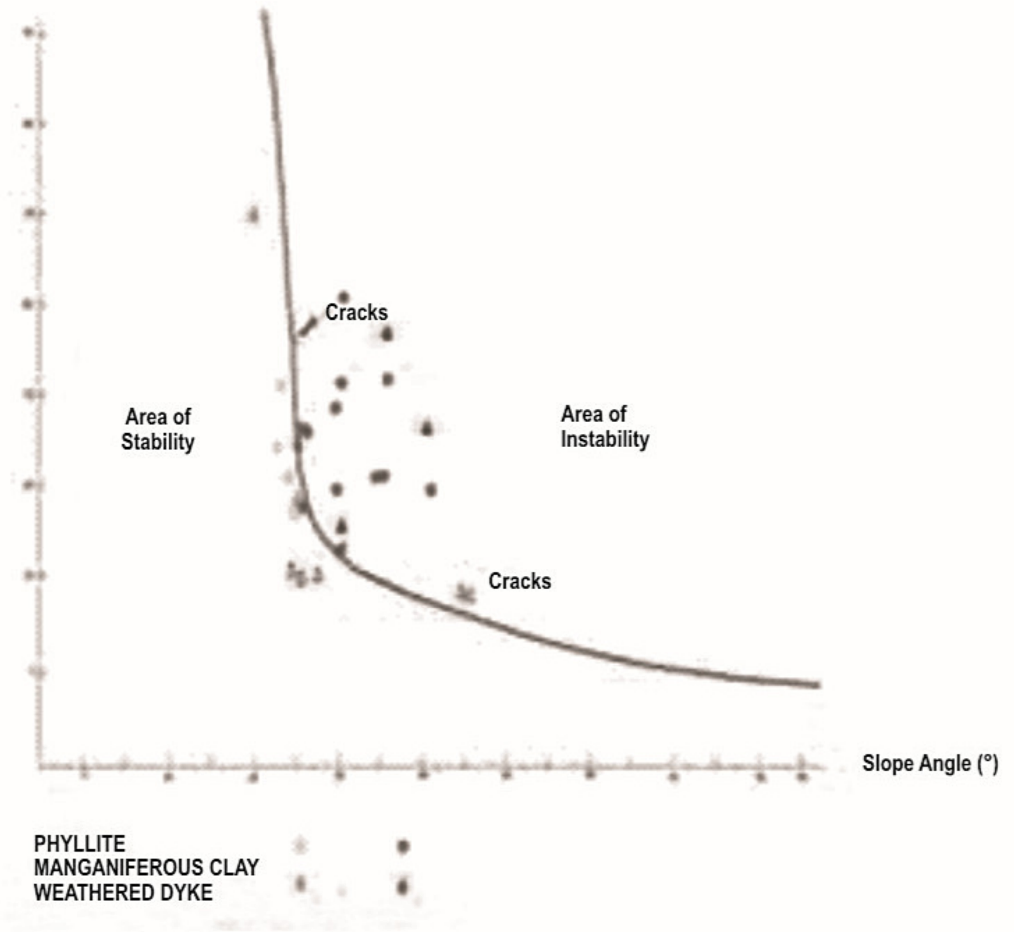


Figure 8.9 - Plot Showing the Slope Failures in Goan Iron Ore Mines

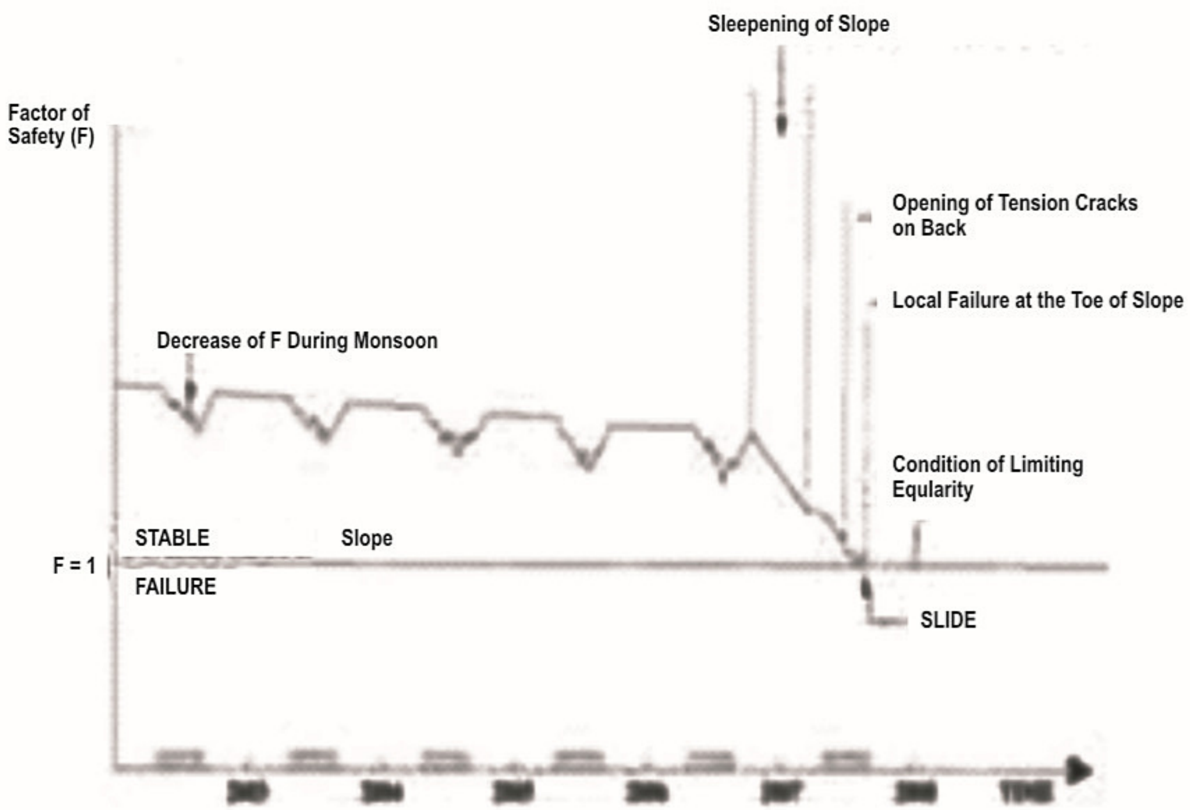


Figure 8.10 - Plot Showing the Effect of Limiting Equilibrium on Slope

Slope design curves for weak rock masses were developed by Robertson et al. (1987), using back analysis of slopes which found that RMR and MS (Hoek – Brown correlation to RMR) were poor predictors of the strength of the rock mass for weak rock masses.

Slope design curves have been developed based on a number of stable and unstable open-pit mine slopes. Shear strength estimates for rock slopes that were proposed by Bieniawski (1976) are too high for values of GSI below 40. The design curves using strength estimates proposed by Robertson (1988) predict steeper angles.

8.2.5 Causes for Slope Failures

The significant parameters causing slope failure in weak rocks are:

1. Shear strength,
2. Slope height and slope angle,
3. Water regime,
4. Pit geometry and irregular shape of the ore body,
5. Geological discontinuities and their properties, and
6. Laterite capping on underlying formations.

The relative importance of these factors is site-specific. For example, shear parameters play a dominant role in soil like ground, whereas the stress regime and discontinuities replace the shear parameters in rock slopes. The role of water regime is more pronounced in cohesive soils, while the water regime as a whole has a subdued influence on rock slope stability. The mechanisms behind these failures are not well known, particularly for weak rock masses. Knowledge of the kinetic behaviour of failing slopes is mostly empirical and requires further research to identify situations in which rapid failures can be expected. The review has shown that the strength parameters of the slope forming materials are heterogeneous in nature, and difficult to assess. However, the interpretation and translation of such data from one geological environment to another, is a difficult task.

8.2.6 Influence of Shear Strength Parameters on Slope Design

Shear strength parameters have a major role in the slope stability of weak slope materials, such as, manganiferous clay, limonitic clay and phyllitic clay etc. due to low values of cohesion and friction angle. Bieniawski (1976) and Robertson (1988) provided estimates of cohesion and friction angle values for different RMR and SRMR ranges respectively for use in slope stability analysis. Robertson's (1988) shear strength correlations were based on the back analysis of failed slopes in weak rock masses.

8.2.7 Effect of Rock Mass on Slope Angle and Slope Height

For the purpose of analysis, the weak rock mass is defined as any rock mass in which the effective shear parameters are less than the following limiting values:

Effective cohesion: c' = 0.2 MPa. Effective friction angle o = 30^{o} using Mohr-Coulomb criterion.

A ground having a combination of shear parameters within the above limiting values would be equivalent to material having unconfined compressive strength of less than 0.7 MPa, a strength normally associated with soil-like material. The results of shear strength properties of Goa formations fall under this category.

A failure surface passing through a weak rock will tend to be selective and pass preferentially through a material which is weaker than the average encountered in any borehole. Thus, it is appropriate to use SRMR value less than the average value. SRMR of the rock mass can be divided into zones of stronger and weaker rock mass strength. Where rating is 40 or higher, it is anticipated that the slope stability will be determined by the orientation and strength along discontinuities. Where the rating is less than 30, failure may occur through the rock mass at any orientation. Tables 8.4 & 8.5 provide a summary of SRMR strength correlations and design slope angles based on SRMR for Goan iron ore mines.

Table 8.4: SRMR—Strength Correlation

Rock Mass Class	Rating SRMR	Strength Parameters Mine-A, Goa		Strength Parameters Mine-B, Goa	
		c (kg/cm^2)	ø	c (kg/cm^2)	ø
IVa	35-40	7.01	16.90	6.21	20
30–35	6.22	9.20	5.52	15.00	16
IVb	25-30	3.90	13.60	4.82	
20–25	3.11	14.00	3.58	22.00	14
Va	15-20	2.85	15.90	2.62	
Vb	15-10	0.60	16.90	0.80	18

Table 8.5: Design Guidelines for Slope Angles based on SRMR System

Rock Mass Class	Rating SRMR	Slope Height (35 m)	Slope Height (45 m)	Slope Height (55 m)	Slope Height (65 m)	Slope Height (75 m)	Slope Height (90 m)	FOS
Iva	35–40	48^0	46^0	44^0	42^0	40^0	38^0	1.21
	30–35	47^0	45^0	43^0	41^0	39^0	37^0	
IVb	25–30	46^0	44^0	40^0	38^0	36^0	34^0	1.22
	20–25	44^0	42^0	39^0	36^0	34^0	32^0	
Va	15–20	43^0	39^0	37^0	35^0	32^0	30^0	1.36
Vb	15–10	37^0	35^0	33^0	31^0	28^0	27^0	1.12
								1.29

The design guidelines are based on SRMR for different rock mass class and the corresponding slope height and slope angle. The plots of normal stress vs shear strength were plotted (see Figure 8.3) from this data and the documented failure for altered rocks for different rock mass classes were plotted. The Factor of Safety (FOS) values for different slope heights and slope angles were derived from results of actual case studies of mines using GALENA computer software.

The design curves are based on charts which assume circular arc failure as observed in the failed slopes in Goa. Bishops simplified method of slice is used for calculation of limiting equilibrium. This is considered an over simplification of the slope and failure geometry. However, more rigorous back analysis of the most recent failures has demonstrated the usefulness and validity of these charts as significant step in slope design. A nomogram has been developed including the SRMR rating, slope height, slope angle and its corresponding Factor of safety. The factor of safety values for four case studies were plotted in the nomogram from which the FOS curves were derived.

8.2.8 Slope Design Nomogram for Weak Rocks

From the data analysis as obtained from Table 8.3 and 8.4, the nomogram is developed to predict to a reasonable degree of accuracy. The nomogram depicts the correlation between slope height, slope angle SRMR values and corresponding factor of safety. The nomogram has been tested for different factor of safety condition in weak rock formations. The factor of safety values were computed from actual case studies utilising the stability plots obtained from the results of numerical modelling. Figure 8.11 is an idealised nomogram for designing different slope angles, with varying slope heights with different SRMR rock mass rating classes.

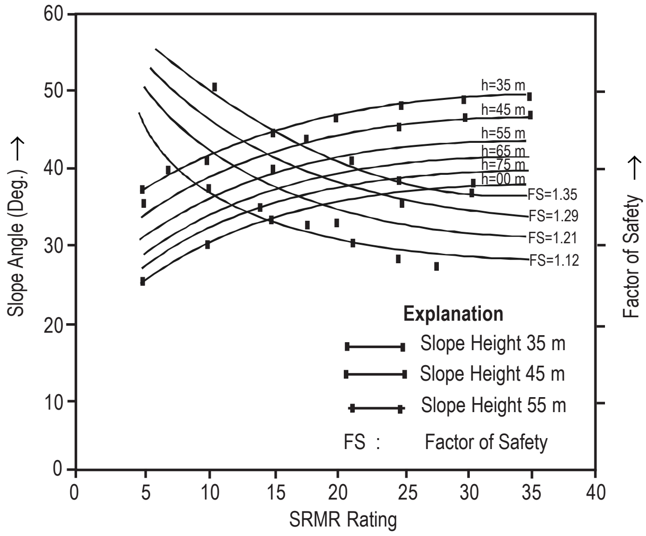


Figure 8.11: *Nomogram for SRMR Vs Factor of Safety*

Based on the analysis of different strength parameters of slope forming materials of documented slope failures, generalised slope design guidelines were evolved for weak rock masses. The nomogram assumes the following conditions:

a) The monogram applies to weak rock/ground slopes

b) Low to very high water pressure condition exists in the slope. The safety factor can be improved by proper drianage methods.

c) The FOS is derived from actual case studies backed by numerical modelling. Usually slope performance curves provide a valuable tool in the design process where rock mass failure plays a strong control in the stability of slopes. The authors included the influence of safety factor as an additional parameter based on the actual case study analyses. Based on the failure modes and analysis of Goan Iron mines, the influencing parameters and its corresponding weight factor percentage were summerised in Table 8.6.

Table 8.6: Weighting Factors for Risk Analysis

S. No.	Parameter	Weightage Factor (%)
1.	Steepening of Slope Angles	45
2.	Laterite Cover and Structural Conditions	25
3.	Influence of Ground Water Pressure	15
4.	Surface erosion of benches	10
5.	Irregular pit design	5

8.2.9 CONCLUSIONS

It is necessary in any slope stability programme to understand the failure mechanisms and the possible conditions under which they may occur. However, our ability to assess different failure types or its combination or dynamic process is limited with the present methodologies adopted. The problem for designers of slope is how to cope up with complexity and variability. The process of design must consider observations on rock behaviour and fracture mechanics, monitoring of rock movements and stresses together with an assessment of the simplified mechanisms of failure to obtain an understanding of ground behaviour. Computer methods can assess the interaction of ground interacting rock failure mechanisms in complex geological conditions. Based on the detailed study of different strength parameters of slope forming materials and analysis of documented slope failures, generalised slope design guidelines were evolved for friable ore bodies.

REFERENCES

[1] Santharam A. and Thote N.R. (2005), Slope Control in Intensely Weathered Rock Masses: Case study of Saniem (Sacorda) Iron Ore Mine, Goa , Proceedings of the National Seminar on Indian Mining Industry (IMID), Nagpur pp. 71–78

[2] Santharam A. and Thote N.R. (2008), Pit Slope Failure Problems in Goan Iron Ore Mines, Goa, India:, 6th International Conference on Case Studies in Geotechnical Engineering, Arlington, Virginia, USA

[3] Santharam A., Dr Thote N.R. and Dr Jethwa J.L. (2010), Applications of Risk Management Techniques for Assessment of Rock Slope Engineering Problems - "Applications of Rock Mechanics –Tools and Techniques". Nagpur, pp. 1–12

[4] Robertson, A. Mac G (1971), Determination of Stability of slopes in jointed rock with particular reference to the determination of strength parameters and mechanism of failure. Ph.D. thesis, University of the Witwatersrand

[5] Robertson A. and Mac G. (1988), Estimating Weak Rock Strength AIME Annual General Meeting, Tucson, Arizona, USA

[6] Serafim J.L. and Periera J.P. (1983), Considerations on the Geomechanics Classification of Bieniawski Int.Symposium Eng. Geology, Lisbon, Vol. 1, pp. 131–142

• • •

APPLICATION OF
ROCK
MECHANICS
IN SURFACE AND UNDERGROUND MINING